JOURNAL OF THE INSTITUTION OF CIVIL ENGINEERS.

No. 3. 1938–39.
JANUARY 1939.

ORDINARY MEETING.

29 November, 1938.

Sir CLEMENT DANIEL MAGGS HINDLEY, K.C.I.E., M.A., Vice-President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

Edwin Massey Bull.
Robert Hartley Heape.
Harold Kerr.
George Matthew McNaughton, B.Sc. (*St. Andrews*).
Egbert Stephen Needham.
William Young Sandeman, M.C., B.Sc. (*Edin.*).
Gilbert Savil Szlumper, C.B.E.
Guthlac Wilson, B.Sc. (Eng.) (*Lond.*).
Ralph Richmond Yerburgh.

And had admitted as

Students.

John Cowper Adamson.
Malcolm Webster Anderson, B.A. (*Cantab.*).
Jack Sloane Berry.
William Brian Harvey Bolt.
Arthur John Bond.
Paul Wilfrid Bott.
Robert Lucien Bourqui.
Arnold Edward Albert Brain.
John George Brown.
James Matthew Campbell.
Arthur Donald Carlaw.
Jack William Carter, B.Sc. (*Bristol*).
Hugh Jenkinson Charlton, B.Sc. (*S. Africa*).
Noel Leslie Costain, B.Sc. (*Birmingham*).
John Clwyd Davies.
Ernest William Davis.
Norman Vernon Dawson.
Peter Deavin.
Peter Edridge.
Douglas Alban St. John Evans, B.Sc. (Eng.) (*Lond.*).
John Findlay.
George Alexander Flook.
Ronald Arthur Aynge Gibbs.
Harry Bartholomew Gifford, B.A. (*Cantab.*).
Alfred Michael Bentley Gray.
William Colin Gray.
Geoffrey Lucas Green.
Jacob Johannes Griessel.
Eric Leighton Haste, B.Sc. (*Leeds*).
Thomas Hedley.
Niels Cecil Højgaard.
John Cyril Hooton.
John Godfrey Howe.

William Howard Jones.
Ronald Ivan Lakin.
Robert Arneil Lee, B.Sc. (*Glas.*).
Arthur David Lindsay.
Archibald Parker Macdonald, B.Sc. (*Dalhousie*).
James Henry Verdun McElhinney.
Amos Ronald Fairlie McGahan.
Leslie John Alphonse Merckx, B.Sc. (Eng.) (*Lond.*).
William Davidson Millar.
Thomas Moore.
George Mould, B.Sc. (Eng.) (*Lond.*).
Broja Bandhu Mukherjee.
John Adams Nauen.
John Newton, Jun.
Raymond Harold Newton.
Reginald Nuttall Norfolk.
Alec George North.
Basil Harold Pattinson, B.Sc. (Eng.) (*Lond.*).
John Sylvester Perkins.
Frank Bernard Pickles, B.Sc. (*Leeds*).
William Pollock.
Kenneth Poyton, B.Sc. (Eng.) (*Lond.*).
Reginald Hugh Prince.
Oliver Thomas Renwick.
Michael Rupert Richards.
Ian Macrae Russell, B.Sc. (*Cape Town*).
Henry Fenton Sanderson.
James Arnold Scott.
Walter Dockray Scott.
Walter Tremelyn Sharp.
Peter Roberts Shimwell.
Sydney Simmons.
Andrew Henry Smith, B.A. (*Cantab.*).
Walter Charles Dudley Smith, B.Sc. (*Cape Town*).
Peter Grainger Stanton.
Peter James Starkie.
Samarth Singh Surana, M.Sc. (*Manchester*).
Norman Sinclair Sutherland.
Geoffrey Louis Tatham.
William Erskine McColl Thomas.
Gordon Urwin.
Ronald Stuart Wells.
Kenneth Ewart Westwood, B.Sc. (*Witwatersrand*).
Donald Wignall.
Edward Unwin Williams.

The following Paper was submitted for discussion, and, on the motion of the Chairman, the thanks of The Institution were accorded to the Author.

Paper No. 5184.

"Improvements at the Royal Docks, Port of London Authority."†

By RALPH ROBSON LIDDELL, M. Inst. C.E.

TABLE OF CONTENTS.

PART I.

HISTORICAL.

THE Royal Docks (Fig. 1, Plate 1) consist of the Victoria dock opened in 1855, the Royal Albert dock opened in 1880, and the King George V dock opened in 1921, each new dock being connected with the older by a passage, thus forming an enclosed water area of 247 acres within a dock estate of ,102 acres. The Victoria dock entrance is 7 miles below London bridge, the distance through to the Albert dock entrances at Gallions reach is 3 miles, and the latter is about 16 miles above Gravesend, where vessels have to stop for H.M. Customs, the Port Medical Officer, and to change pilots.

In order to appreciate the scope of the works and improvements carried out by the Port Authority, it is necessary to give some particulars of the two older docks and the conditions which obtained when they were taken over in 1909.

† Correspondence on this Paper can be accepted until the 15th April, 1939.—EC. INST. C.E.

The construction of the Victoria dock was described in a Paper to The Institution by Mr. W. J. Kingsbury, Assoc. Inst. C.E.*; the main features being a main dock of 74 acres and a tidal basin of 16 acres, all cut out of the marsh land, the surface of which was about 8 feet 6 inches below Trinity High Water and protected by river banks to 5 feet above that datum. On the north side four solid piers were built, each 500 feet long by 140 feet wide and containing a two-storey brick warehouse 80 feet wide with vaults. Later several intermediate timber jetties about 400 feet long by 80 feet wide, and considerable accommodation in the way of sheds, warehouses, granaries, and refrigerating chambers, were added.

The length of quays available for berths was 11,740 feet and, in addition, 3,030 linear feet was leased to various firms for flour-mills, coal-wharves, etc.

The depth of this dock and basin and the inner sill of the lock was 25 feet 6 inches below Trinity High Water, the lock itself being 325 feet long by 80 feet wide with a depth of 28 feet below Trinity High Water. As the depth of water over the sill at neap tides was 24 feet any vessel requiring more than that had to wait for spring tides. These conditions prevailed for 25 years until the opening of the Royal Albert dock, when steam pumps were installed at the east end of the new dock to maintain the water of both docks at the level of Trinity High Water.

The Royal Albert dock, completed in 1880, consisted of a main dock of 73 acres, 6,600 feet long with a uniform width of 490 feet and a depth of 27 feet below the impounded level of Trinity High Water; a passage at Manor Way 80 feet wide and 27 feet 3 inches deep; and a basin of 14 acres leading to a lock 550 feet long by 80 feet wide and 30 feet deep below Trinity High Water. The depths at the passage and the lock were reduced by the inverts rising 4 feet 6 inches to the side walls, which corresponds to 3 feet for ships of 70-foot beam. The limited depth no doubt led to the construction and completion of the lower lock in 1886 with a depth of 36 feet below Trinity High Water, again less 3 feet for a 70-foot beam ship. The Royal Albert dock was also provided with two dry docks, the larger being 500 feet long and 62 feet 9 inches wide at the springing of the invert. The total length of quay available for berths was 15,160 linear feet.

An obligation in the construction of the Royal Albert dock was to transfer the North Woolwich branch railway line of the Great Eastern Railway from the surface at Connaught road to twin tunnels under the new 80-foot wide passage. The distance to the intrados of the tunnel-arches was fixed at 30 feet and the depth over invert of the new passage at 25 feet 6 inches, both below Trinity High Water.

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* "Description of the Entrance, Entrance Lock, and Jetty Walls of the Victoria (London) Docks; with a detailed account of the Wrought-Iron Gates and Caisson, and remarks upon the form adopted in their Construction." Minutes of Proceedings Inst. C.E., vol. xviii (1858–59), p. 445.

In 1902 a Royal Commission reporting on the condition of the river and docks stated :—

"It is obvious that the Dock Companies could gain no advantage in deepening the entrances of their docks if the river remained unimproved, because, as is well known, large ships, with the river in its present condition, can only approach or leave the Docks at or near the time of high-water and at this time of tide the locks, although perhaps susceptible of some improvement, are not conspicuously inferior in depth to that of the channel of the river at such times."

The limiting depth of water in the river at low water spring tides was 16 feet above and 25 feet below Gravesend.

The Royal Commission recommended that a channel of not less than 30 feet in depth at low water spring tides should be made from the Nore to the entrance of the Royal Albert dock at an estimated cost of £2,500,000, and improvement and extension of the docks at an estimated cost of £4,500,000 ; a total sum of £7,000,000 to be raised and expended within 10 years.

The above represents the obligations taken over by the Port of London Authority when constituted in April 1909.

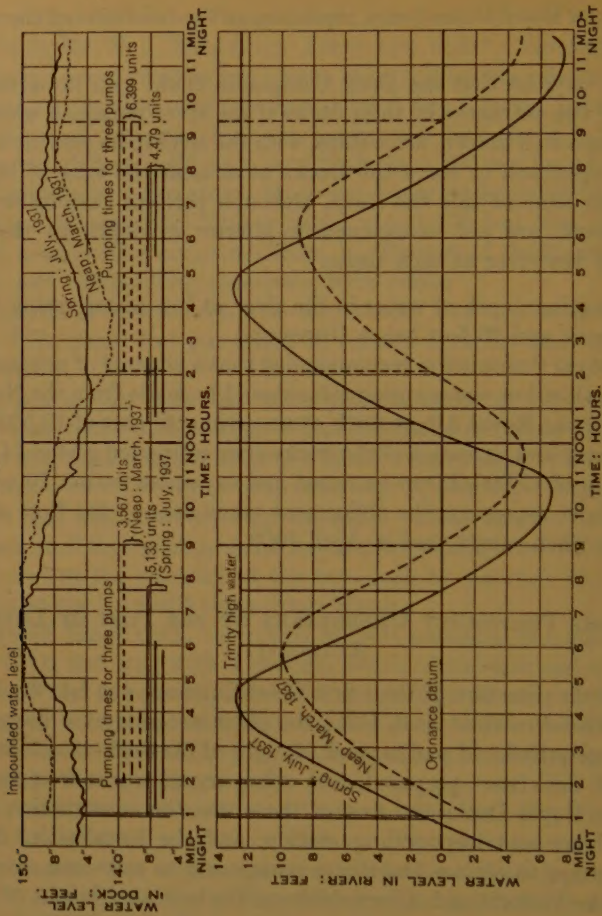
Works Previously Undertaken by the Port of London Authority.

Under the direction of their first Chief Engineer, the late Sir Frederick Palmer, Past-President Inst. C.E., the plant necessary for deepening the river was acquired and put to work, designs of the dock-extension south of the Albert dock were prepared, and other urgent works were started.

At the Royal Docks the first of these was the construction of a new intake-culvert and impounding station, and the installation therein of electric pumps to replace the steam pumps and to raise and maintain the water in the Victoria and Albert docks to the increased height of 30 inches above Trinity High Water. This was completed in 1911 at a cost of £44,500. The culvert, 13 feet 4 inches wide and 14 feet 3 inches deep, is 400 feet long between the 30-foot wide bellmouth at the river-side and the sump (80 feet long by 21 feet wide). The invert is level throughout at 23 feet 6 inches below impounded level, or 3 inches above low water spring tides.

The three pumps were designed to discharge 15,000 cubic feet per minute each, during a period of 3 hours before to 3 hours after high water against a head varying from 15 feet to 2 feet 6 inches. Each pump has one 70-inch suction and two 50-inch discharge-branches connected together into one 70-inch outlet. The suction- and discharge-pipes are each bellmouthed to 95 inches. Each main pump is driven by a 2-phase 50-cycle 6,000-volt slip-ring induction-motor, designed to develop 430 brake horse-power at 195 revolutions per minute.

Figs. 2.



VARIATION OF DOCK- AND RIVER-LEVELS.

The diagram (*Figs. 2*), recorded electrically, shows the variation of the dock- and river-levels during 24 hours of neap tides in March, 1937, and spring tides in July of the same year. This diagram also shows the starting and stopping times of each pump and the total units used during the tides.

In 1912 work was commenced on the major improvement, namely the construction of the King George V dock, which was described in a Paper to The Institution by Mr. Asa Binns, M. Inst. C.E.[1]

This dock, 65 acres in extent, with 38 feet depth of water (Fig. 1, Plate 1), was joined to the Albert dock by a passage, 100 feet wide and 34 feet deep, which, since 1921, has made access to the older docks possible for vessels

[1] "The King George V Dock, London." Minutes of Proceedings Inst. C.E., vol. ccxvi (1922–23), Part II), p. 372.

up to 650 feet in length and, as the depth over the sills of the new lock is 45 feet below Trinity High Water, the period over which locking was possible for deep-draughted vessels using those docks was considerably increased.

At the same time a contract was placed for widening the western dry dock from 56 feet 6 inches to 80 feet and for lengthening it from 408 feet 6 inches to 575 feet; a new steel rectangular caisson was also built, inside the dock. The old hardwood keel-blocks 42 inches high were replaced by cast-iron wedge blocks with greenheart caps and, at the shipowners' request, the height was increased to 54 inches. Thus, although the depth over the sill had been increased to 25 feet by impounding, the depth over the keel-blocks was limited to 23 feet 6 inches. This work was completed in July, 1914, at a cost of £67,000.

The steam pumping plant serving the two dry docks was replaced in 1917 by two 34-inch Tangye pumps driven by two 220 brake horse-power, 250-volt synchronous motors at a speed of 250 revolutions per minute, and capable of de-watering the enlarged dock in 3 hours. In 1920 four sets of motor-driven air-compressors were installed for 100 tools at 100 lb. per square inch pressure, and a 25-ton electric travelling crane and track provided for each dock.

On the north quay of the Albert dock and basin a crane track of 13 feet 6 inches gauge founded on 15-inch "Simplex" piles at 16-foot centres, with an electric conduit, was constructed for a length of 7,020 feet; and by 1916, forty-three new electric cranes of 3 tons lifting capacity at 60 to 65 feet radius had replaced the 30-cwt. 40-foot radius hydraulic cranes at a cost of £85,000.

At the west end of the above quay three of the transit sheds were replaced by a two-storey reinforced-concrete shed 1,100 feet long by 110 feet wide on the upper or cold-sorting floor and 123 feet 6 inches wide on the quay or transit-floor. This building is connected by two steel conveyor-bridges across the road and railways, to the top floor of a new cold store (No. 6), a six-storey reinforced-concrete building divided vertically into four sections, insulated throughout by two layers of 3-inch cork slabs and arranged with air-ducts for circulating cold air in any section. The cold store was completed in 1918 and the cold sorting floor in 1920, at a total cost of about £450,000, and increased the cold-storage accommodation (15° F. to 18° F.) at these docks from 2 to 4 million cubic feet, representing a holding capacity of nearly one million carcases of mutton.

In 1918 the exchange sidings along the landward side of the Royal Victoria dock were remodelled and extended westward; sixty-four new turnouts and 6,000 yards of sidings being laid down in 75-lb.-per-yard flat-bottom rails, this being the standard section adopted in the docks.

In 1920 the accommodation for the warehousing of tobacco was increased by 580,000 cubic feet by the erection of a single-storey brickwork building, wherein American tobacco in hogsheads weighing about 10 cwt. are handled

and stored by means of electric overhead travelling-cranes, which pile th hogsheads in tiers five high. This was followed by the completion in 192 of a six-storey building (shown as "M" in Fig. 1, Plate 1) built c reinforced-concrete with brick-panel walls and concrete floors and equippe with two 2-ton electric lifts, hydraulic baling presses, and nine 30-cwt electric wall-cranes for receiving the tobacco from craft alongside the nev reinforced-concrete quay or for delivering this material to rail and roa vehicles on the landward side. This warehouse added 1½ million cubi feet to the tobacco accommodation at a cost inclusive of quay, railways etc., of £230,000.

The success of the storage of hogsheads 5 tiers high led to the recon struction on similar lines of two old tobacco warehouses in 1928.

To provide for the rapid development of the chilled-meat trade whicl had been located at shed No. 35, Royal Albert dock (Fig. 1, Plate 1 since 1921, and the increase in size of the steamers engaged in the trade, new reinforced-concrete quay 600 feet long, on cylinders, was constructed o the north side of the Tidal basin of the Royal Victoria dock and a new she erected. This is equipped with 6,000 feet of overhead mechanical runway on which the quarters of beef are hung and travel over automatic weighin machines to insulated road or rail vehicles waiting at any position alon the 1,450 linear feet of covered-in loading platforms. This work wa completed in 1926 at a cost of £146,000. The average quantity of mea discharged from the vessel arriving each week is about 3,800 tons.

To provide a second berth for this trade, 600 linear feet of the timbe quay at the east end of the Royal Victoria dock was reconstructed i reinforced-concrete, and "A" Shed (Fig. 1, Plate 1) was erected an equipped with runways, similar to the above, serving 1,500 linear fee of loading platforms. This work, which was completed in 1928 at a tota cost of £123,000, involved the demolition of No. 2 cold store erected i 1890 for frozen meat.

In 1934 two more of the timber-built cold stores—described in a Pape to The Institution by the late Mr. H. F. Donaldson, M. Inst. C.E.[1]—wer demolished, No. 1 to provide space for 1,000 linear feet of covered platform with motor-driven conveyor-belts for the delivery of bananas from th quayside elevators to road and rail vehicles, and No. 3 for the erection of single-storey reinforced-concrete shed on "Vibro" piles, with a roof designe as a floor for future requirements. This shed provides a floor area of 40,00 square feet with a height of 13 feet 6 inches for the sorting and storage o oranges and other fruit, and has 500 linear feet of covered platforms fo deliveries to road or rail vehicles. It was completed in 1935 at a cost o £66,000.

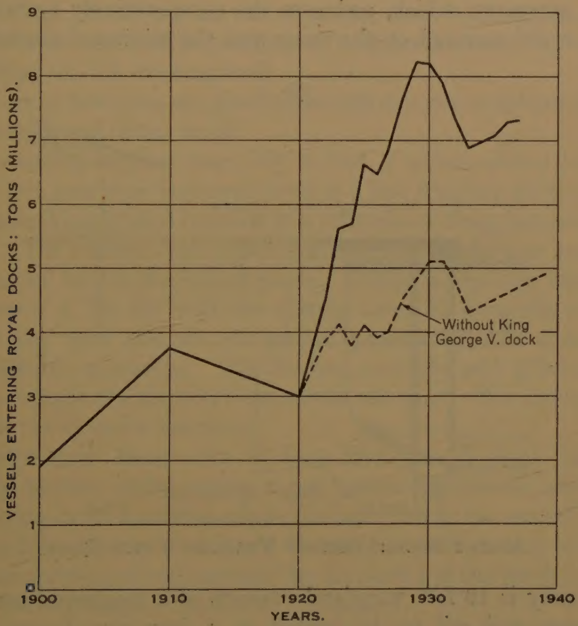
As the Port Authority, about this time, had under consideration furthe

[1] "Cold Storage at the London and India Docks." Minutes of Proceedings Ins C.E., vol. cxxix (1896–97, Part III), p. 1.

evelopments, it is interesting to note here that the capital already laid ut on works at this group up to March, 1935, amounted, in round figures, o £6,000,000, which, added to the 1909 value of £8,000,000, brought the otal capital value of the Royal Docks to £14,000,000.

The accompanying graph (*Fig. 3*) shows the rapid increase in the annual et registered tonnage of vessels entering these docks during the above eriod, and that even during the 1933 slump the tonnage was about double he 1910 figure.

Fig. 3.



TONNAGE OF VESSELS USING ROYAL DOCKS.

The size of the vessels using the docks was also increasing and it was evident, by the numbers of vessels which had to lighten in the George and Albert docks, before proceeding to their usual berths in the Albert and Victoria docks, respectively, that draught had overtaken the extra depths provided in 1911 by impounding.

In consequence the Port Authority decided to increase the depth of the Albert docks by dredging from 29 feet 6 inches to 34 feet, the latter being the depth in the passage from the George dock ; to construct a false quay at that depth along the greater part of the north quay of the Albert dock ; and, as a large and increasing proportion of the vessels using the Port approach 29 feet in draught on arrival, they also decided to undertake the

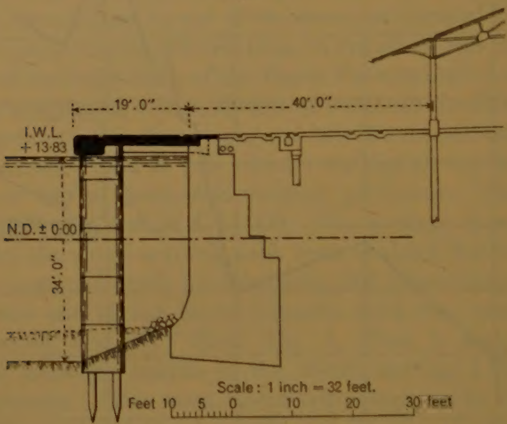
deepening of the passage to the Victoria dock to 31 feet, and to remod the accommodation in that dock with an increase in depth to 31 feet, a under powers obtained by Act in 1935.

PART II.

Widening North Quay, Royal Albert Dock.

In order to deepen the Royal Albert dock to 34 feet withou undermining the foundations of the quay-wall, the construction of a fals quay was necessary, which, owing to the comparatively narrow width c the dock, would encroach on the water area the minimum amount possibl

Fig. 4.



Albert Dock: Section Through North Quay.

The new quay is 19 feet wide and consists of a reinforced-concrete de for the crane and rail tracks carried on a deep cope-beam, bearing cylinders spaced at 24-foot centres, and tied into the existing quay transverse beams at 8-foot centres. (*Fig. 4.*)

Work started at both ends simultaneously in April, 1935, the procedu being as follows :—

After grabbing the dock-bottom to remove obstructions, steel gui frames were lowered on to the dock-bottom and secured in the cylind positions by a whole-timber horizontal frame at water-level. Pre-ca cylinders each 8 feet long were then lowered through the guides and su into the foundation-strata by grabbing. Two 14-inch square piles 50 f long were then driven within each cylinder, after which the cylinders w hearted to water-level with 1-to-4 ballast concrete deposited continuou by boxes with top and bottom folding doors lowered through the wat At the west end a layer of septaria was encountered, and had to be brok

o by chisel-pointed steel joists used as rockbreakers. Towards the east d the cylinders had to be sunk through a hard layer of sand by divers ndercutting with compressed-air spades.

Owing to the low headroom between the water level and the quay-rface, the erection and striking of shuttering for the decking presented nusual difficulties which were overcome by the adoption of the following ethod :—

Near the top of each cylinder the contractor built in a length of heavy ction rail with a short projection on each side parallel to the cope ; on hese rails special steelwork brackets were hooked to form bearings for eep steel girders placed outside the cope-line, from which hinged shutter-ng was suspended and held in position by long links secured by cotter pins bove bearings on the existing wall.

A set of seven brackets, six girders and 120 feet run of hinged shuttering as used at each end of the work.

As the existing coping was only 3 feet 3 inches above impounded ater-level, the new cope-level was fixed at 3 feet 6 inches above this level, nd the new decking graded inwards to a continuous drain formed along the ld coping. The electric cranes, when transferred to the new crane track, ere re-adjusted to suit the inward grade. Owing to the very flinty nature f the concrete of the old wall the cutting away to form the continuous earing for the deck and the dovetailed pockets for the anchor-beams roved a difficult operation. The broken concrete and pile-heads were eposited to weight the slope in front of the old quay. No displacers were ermitted in the cylinder hearting.

The contractors, Messrs. Sir Robert McAlpine & Sons, by working ontinuously in two shifts, using rapid-hardening cement, and striking offit shuttering in 72 hours, succeeded in completing the overall length of ,450 feet in 39 weeks, an average of 70 feet per week at each end.

On the completion of each section the deepening of the berth was carried ut by the Port of London Authority's dredgers.

The total cost of the work was £127,000, of which £42,000 was for redging.

Works at Connaught Road Passage.

ntroduction.

Since the increase in the depth from 25 feet 6 inches to 28 feet by mpounding, this passage had been the main entrance to the Victoria ock for all shipping, and in 1930 the gates at the western entrance, which ad become very troublesome to maintain, were reduced in depth and new ills formed 20 feet 6 inches below Trinity High Water for the convenience f barge traffic.

The Connaught Road passage is intersected by twin railway-tunnels arrying the Fenchurch Street and North Woolwich branch-line of the

London & North Eastern Railway at a depth which provided a cover r only 4 feet 6 inches thickness of brickwork over each tunnel. The reducti of this cover to 1 foot 6 inches in order to deepen the passage to give depth of 31 feet required extreme caution. The twin tunnels are 120 yar in length between the ventilating shafts north and south of the passag The total length of the tunnel section of this line is 600 yards, and t seepage and drainage from this and the open approaches, amounting about 400 gallons per minute, is pumped by the Port Authority at the nor side of the passage. The London & North Eastern Railway have the righ in an emergency, to pass their traffic over the Port Authority's high-lev railway which crosses the passage by a swing bridge.

Early in 1935 the Railway Company approved a scheme to lower t railway lines and to strengthen the tunnels with cast-steel linings prior the removal of 3 feet of the brickwork cover.

A contract (Figs. 5 and 6, Plate 1) let to Messrs. Chas. Brand Sons, Ltd., in August, 1935, comprised the following :—

(i) The strengthening by cast-steel lining, for a length of appro mately 160 feet, of each of the twin tunnels.

(ii) The construction eastward of the tunnels of a cast-iron subw under the passage with two cast-iron shafts for the diversion mains.

(iii) The deepening of the passage by removal of brickwork, et over the twin tunnels, existing culverts, and pipe-trenches.

Tunnel-Strengthening.

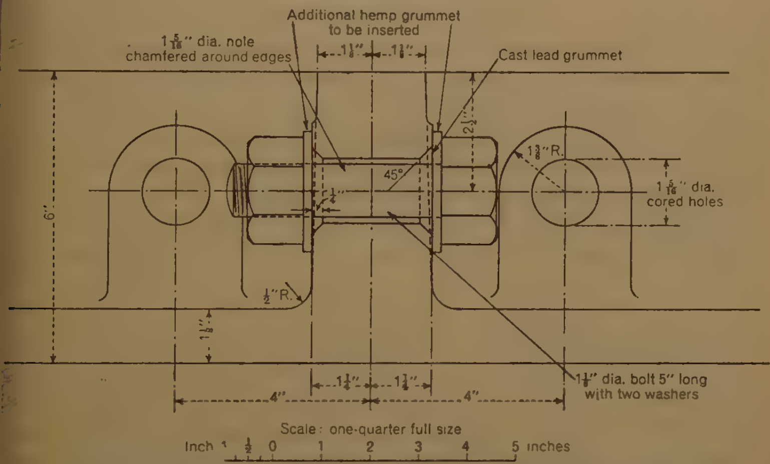
Before commencing work on the "down" tunnel, the Railway Compa altered the approach lines and signalling for single-line working on t "up" line, the high-level lines were reconditioned to take passenger-traff and the Port Authority had arranged with the shipping companies f suspension of shipping traffic through the passage during the morning a evening in order that the swing bridge should be available for passeng traffic.

On making inquiries concerning tunnel-linings it was found that Great Britain there was no previous experience available with regard the use of cast-steel segments for this purpose. It was considered that t ideal method would be to have each ring assembled complete before t machining of the circumferential joints, but the prices quoted and ti required to deliver were prohibitive. An offer was eventually obtain and accepted in March, 1935, for the supply of 600 tons of cast-steel s ments (Fig. 7, Plate 1) separately machined to a tolerance of $\frac{1}{64}$ in with delivery in 6 months. The difficulties encountered in the manufact of the segments such as warping and honeycombing, and in machining, w successfully overcome, but the tolerance was not always obtainable, chie due to varying hardness of the metal.

The cast steel was in accordance with B.S.S. No. 30 "Castings for rine Purposes" of quality 26 to 35 tons per square inch tensile strength. e thickness was $1\frac{1}{8}$ inch in the skin and $1\frac{1}{4}$ inch in the flanges. All nts were machined to the full depth of the flanges. The bolt-holes, inch larger than the bolts and slightly countersunk, were drilled after chining. The segments were all 24 inches wide and the heaviest weighed out 9 cwt.

The south ventilating shaft was used as the sole access to the tunnels d the contractor erected his plant and shops in its vicinity. The rmanent way and ballast having been removed and the drain diverted, e brick benchings were cut away, each side wall cut back an average

Fig. 8.



Detail of Circumferential Joint for Cast-Steel Segments.

pth of 7 inches to the new profile, and the invert and soffit of the gmental arches dressed over to leave a clean surface and a 1-inch space r grouting. The brickwork of the 4-foot thick division wall was cut rough to form two refuges measuring 4 feet by 8 feet. The overall mensions of the rings were 17 feet 1 inch on the vertical axis and 15 feet inches on the horizontal axis. The flanges were 6 inches wide.

Frequently, before erection, the segments for two adjacent rings were sembled in complete rings and bolted up to check the accuracy of the ork and the efficacy of the fastenings. As a result, the bolts were creased to 5 inches long and several were specially drilled and tapped to able the jointing to be tested under hydraulic pressure. These tests owed that a water-tight arrangement was obtained by using a single and of red-leaded hemp between the washer and the lead grummet *ig. 8*).

The erection of the segments commenced at the centre on the 7 October and proceeded in both directions; usually three rings at a ti were grouted with neat cement, and finally all joints were electrica seam-welded by a specialist firm. Following this welding the test bo referred to above were inserted in flanges which were not a close principally in the upper half of the lining, and a special portable hand pur was used to force a liquid red lead at a pressure of 90 lb. per square in in the wide joint against the back of the welding. It is satisfactory record that only in very few instances was it possible to discern the r lead on the surface of the weld.

After an interval for the completion of the permanent way, alterati to temporary works and complete delivery of the segments required, traf was changed over to the "down" tunnel and work commenced in t "up" tunnel on the 12th January. Erection commenced at the specifi ring opposite the north refuge and, proceeding on the same lines as for t "down" tunnel, was completed on the 28th March, 1936.

In each case the inside of the tunnel-lining was coated with tar in accor ance with the following specification: to 1 gallon of heated coal t 3 ounces each of Russian tallow and unslaked lime were added; t mixture was then well stirred and applied hot.

This section of the work cost £27,200 or £85 per foot run of steel lini The high- and low-level track-work cost £13,000 in addition.

Pipe-Subway under Passage.

The new subway (Figs. 5 and 6, Plate 1) was constructed accommodate the public and dock mains carried across the passage three brick culverts which were affected by the deepening of the passa The centre-line was fixed at 55 feet east of the tunnel centre and the sou shaft at 50 feet normal to the quay-wall to allow of possible widening the passage. Borings taken on the shaft-sites indicated a suitable stratu of blue clay with shells at 9 feet below the level for the deepening of t passage.

Each shaft is 15 feet in external diameter and consists of thirty-fi rings, each 20 inches deep, from impounded level to the bottom of the seal —44·82 N.D. The cast-iron segments are 1 inch thick with flanges 6 incl deep and 1¼ inch thick after machining. The rings were held togetl by fifty-five ⅞-inch diameter bolts, 5 inches long, fixed through 1⅛-inch co holes. Red-leaded hemp grummets were used under the mild-steel wash and 15 per cent. of the bolts were 6½ inches long to take hangers. T caulking groove, 1¼ inch deep by ¼ inch wide, was caulked with rou lead.

The south shaft was sunk first. Under a timbered trench 17 feet d three rings, Nos. 9, 10, and 11 from the top, were built and concreted over these the steel-framed air-deck was formed and suitably weighted

he full upward air pressure, and the 9-foot air shaft erected with a 6-foot -inch air-lock a few inches above surface level. Rings Nos. 12 to 35 were hen excavated, built in and grouted under air pressure reaching 20 lb. er square inch in the south and 17 lb. per square inch in the north shaft. ach seal consisted of 7 feet of 1 : 2 : 4 concrete placed in two layers, the ower being 15 inches thick from 4 inches below the bottom flange, and when et this was covered with three layers of asphalt tucked under the upper lange of the bottom ring. No trouble was experienced in sinking the shaft n the north side, but in the south shaft a disused brick sewer and a timbered rench were encountered ; also, air escaped and was found blowing as far field as the approach to the tunnel, about 500 yards north of the shaft, nd it was not until two 15-inch drain pipes found under ring No. 32 had een plugged and sealed that sinking proceeded normally. These two drains ay across the shaft parallel to the passage, and were evidently relics of he tunnel construction in 1879.

The subway is 8 feet 1½ inch in external diameter and consists of inety-three rings each 21 inches long, between the shafts, the centres of which are 174 feet apart. The horizontal axis is 47 feet 6 inches below mpounded-water level. The cast-iron segments are ⅞ inch thick with 1¼-inch flanges, 1¼ inch thick after machining, and they were bolted ogether by ¾-inch bolts 4½ inches long, 25 per cent. of these circumferential olts being 6 inches long to take hangers. The caulking grooves were ¾ inch deep and ¼ inch wide, and were filled with round lead.

Driving started at the south end in air at a pressure of 20 lb. per square nch, and by working three shifts per day continuously, reached the north haft in 22 days. A layer of septaria on the axis of the subway between he blue clay and the green sand somewhat delayed progress ; these lime-tone boulders were very hard, and some of them, projecting more than alf-way across the face, had to be cut through and broken up for removal.

At each end a collar of fine concrete was packed round the subway ron prior to lead caulking the space, averaging 2 inches wide, round the ircular flange of the " picture frame " or shaft lining. Nearly 100 tons of ement was used for grouting the shafts and subway.

When the mains were removed from the existing culverts a 14-inch rick stunt-head was built across each, under the face line of the quay-wall, nd the shaft end was sealed up with 8 : 1 concrete to 1 foot above the crown f the culvert ; the shafts above this level were then filled in.

This section of the work cost £9,600 or £35 per foot run of centre-line f subway and shafts, exclusive of £1,700 for mains.

Deepening of Passage.

The passage (Figs. 5 and 6, Plate 1) is about 340 feet long by 80 feet wide and the undertaking was, on the completion of the tunnel-lining and iversion of mains to the new subway, to remove 3 feet of the brick and

concrete work over the 100 feet length of the tunnel and culverts, and dredge the remainder of the passage, all without delaying the swinging the bridge for vessels.

The stages of the work carried out were as follows :—

(1) 28th November, 1935, to 27th February, 1936 (13 weeks). Dive cutting trench in front of footings and dressing off "toe" of quay-wal clear of tunnel area.

(2) 27th February to 20th June (17 weeks). Six divers cutting separa chases in the brickwork over the tunnels with compressed-air tools. Th output averaged 1 cubic yard per diver per week, but the brickwork an mortar were very hard and the loss of time due to craft and shippin amounted to 33 per cent. Drilling 5-inch holes 12 inches apart, ahead the divers, from a pontoon moored over the site of chases, did not improv the rate of progress, as one diver was required to attend on the dril The total cost of one hundred and thirty-four holes 30 inches deep wa £455. A hydraulic cartridge was tried in similar holes in a large concret base but the results were considered too indefinite for its use near th tunnel.

(3) 20th June to 12th July (3 weeks). A floating rockbreaker with 10-ton ram, 18 inches diameter with a shell-pointed end, was employed t crush the concrete over the culverts on each side of the tunnel. In th first cut, holes 3 feet apart each way were penetrated to a depth of 4 fee When the debris had been cleared away by grab-hopper the areas we pounded a second time with holes at the same spacing but intermedia to the former. Owing to the length of the pontoon of the rockbreaker strip from 8 to 10 feet wide at the quay-walls was inaccessible. Measur ments showed that 330 cubic yards had been broken up to an avera depth of 3 feet in 12 days of work carried on continuously except for traff delays. In addition a concrete pipe-chase 12 feet wide and 4 feet thic was broken up for a length of 62 feet (equal to 80 cubic yards net) durin a week-end when the swing-bridge was closed to road traffic. The co of this crushing, exclusive of removal, was about £2 per cubic yard.

The demolition of this brickwork required far more violent treatme than was anticipated, and instead of the culverts collapsing when heavi pounded by the rockbreaker, the ram punctured on one occasion the out culvert on the east side and stuck fast for hours, delay to shipping bei obviated by removing it with a 150-ton floating crane.

It should be mentioned that while this work was in progress the si was "swept" clear to the former depth by a "drag" towed by a sma tug belonging to the Port Authority before the passage of each lad vessel.

(4) 2nd July, 1936, to 30th January, 1937 (30 weeks). During th period the thickness over the tunnels was reduced to about 18 inches miners working in a diving bell, the compressed air for both tools and b being supplied by a 3-inch main laid from the dry docks.

In the first instance a small cast-iron bell belonging to the Clyde Trust as used ; this weighed about 10 tons and was slung over the end of the uthority's 30-ton floating crane moored alongside the north wall and onnected to the air-main with two 50-foot lengths of 3-inch flexible hose. he cuts were about 12 inches deep and were taken out in strips 5 feet wide y the crane heaving along the wall as required and slewing out for each ew strip. On completion of the second 12-inch cut over the north half, e crane was transferred to alongside the south wall and worked over the outh side as above. The third and final 12 inches were taken out by a rough ut of 9 inches two strips in advance of a final cut of 3 inches ; the depth r the final cut was measured from a straight-edge suspended in the bell t a fixed distance from impounded-water level, any variation in which as communicated to the miners by telephone.

The work was carried out continuously in three 8-hour shifts, from 0.30 p.m. on Sundays to 2.30 p.m. on Saturdays ; that is, seventeen shifts er week. The small bell was 6 feet 5 inches long by 6 feet wide ternally, and was manned by two miners per shift, who took turns in rilling and collecting the broken material. They were paid 21*s.* per shift lus a bonus of 1*d.* per bucket for the first thirty buckets per shift and *d.* per bucket thereafter, and it was agreed that ninety-eight buckets had o fill a box measuring 44 cubic feet. 1 cubic foot of this material weighed 3½ lb., whereas it weighed 125 lb. when solid. The output from this ell averaged 15½ cubic yards per week over the 14 weeks it was in use.

With a view to doubling the output a steel bell was constructed 10 feet inches long by 6 feet 6 inches wide to accommodate four miners, and as equipped with a 2-inch air-pipe and valves for three air-tools and also ur longitudinal seats to slide into position as shelves to receive the broken aterial. This bell was used by the three shifts for 9 weeks and averaged ist under 21½ cubic yards per week, all from the third cut. It was also sed on single shifts for 7 weeks, skimming over and removing "pimples" om the whole area.

The rectangular pontoon of the floating crane with a beam of 41 feet eltered the bell and enabled the miners to work in safety whilst leaving nple room for tugs and craft to pass in single file, thereby eliminating, hen working clear of the radius of the swing-bridge, 19½ of the 33 per nt. delays previously referred to. The weight of the steel bell was 15 tons, it when loaded the weight on the hook of the crane, inclusive of sling ains, was 20 tons. A diver and linesman were in charge of the diving ll day and night and the telephone proved reliable and useful, but signals om the miners to the crane were made much quicker by push bell. With e aid of two electric lights fixed in the bell every joint in the brickwork uld be seen, and when accustomed to the difference in pressure the work- g conditions inside the bell were ideal for the job. By this means 440 cubic rds solid were cut and removed, at a cost of £17 per cubic yard inclusive the working costs but not of the hire of the floating crane and compressors.

(5) 30th January to 9th April, 1937 (10 weeks). During this period tw divers were employed dressing off the brickwork at the junction with tl quay-wall for a length of 100 feet on each side of the passage. A ste template bent to a radius of 6 feet and suspended from the coping to tl required depth was used for this purpose.

In the period covered by the deepening it was necessary to close tl swing-bridge to rail and road traffic for ten week-ends during which pr vision had to be made for pedestrians and perambulators to cross the passa by pontoon and gangways, which had to be removed for, and replace after, each vessel.

The total cost of this deepening, including dredging and grabbing tl full length of the passage, amounted to £17,500, which, added to the co of the two other sections, made the total £70,000.

New South Quay and No. 1 Shed.

Although the sheds and buildings on the south side of the Victoria do were rebuilt on modern lines with ample road and rail facilities after tl disastrous Silvertown explosion in January, 1917, the only usable qua was that in front of Sheds Nos. 2 and 3 (Fig. 1, Plate 1). This quay, bui in 1892, was about 1,000 feet long and consisted of several rows of timb piles supporting steel joists and troughing covered with concrete deckin The width varied from 59 to 67 feet and had a sharp gradient up to tl level of the shed floors, aggravated by the deterioration of the timb campsheeting retaining the ground in front of the new sheds; it wa inconvenient for working and, moreover, was not strong enough to tal the weight of modern cranes.

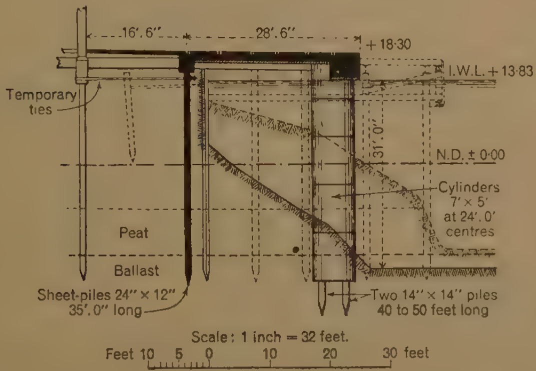
The Port Authority let a contract in August 1935, (*a*) for the replac ment of this quay in reinforced concrete, (*b*) for the removal of the sol jetty separating the south side of the basin from the main dock, (*c*) fo the extension of the concrete quay across the basin to a total length 1,750 feet, and (*d*) for the foundations for a new shed.

The work started at the east end of No. 3 shed by the excavation of trench down to the tie-rods of the old campsheeting which were cut as tl driving of the 24-inch by 12-inch concrete sheet-piles advanced; tl latter were held by steel channel walings tied back by 1½-inch rods the existing shed-columns during the construction of the new quay.

A 5-ton steam crane running on a track laid along the outer portion the existing quay served for the construction of this length (*Fig. 9*). Hol were cut in the decking, piles drawn, and the cylinders sunk through ste guide frames from level benchings formed at −5·00 N.D. by grabbin All material was delivered by water to a floating mixer-plant and, aft piling and hearting the cylinders, the inner portion of the existing quay wa removed. Pre-cast soffit beams (Figs. 10, Plate 1) were then place between the cylinders spaced at 24-foot centres, on which the shutterin

or the coping beams was erected. The steel of the main transverse beams at 24-foot centres was continued through the sheet-pile capping beam and lap-welded on to the bars of the anchor tie-beams. These were provided

Fig. 9.

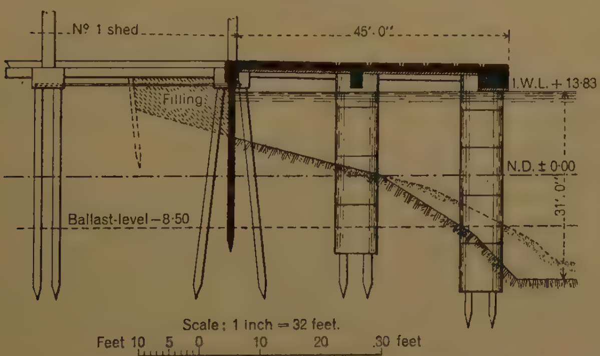


VICTORIA DOCK: SECTION THROUGH SOUTH QUAY AT NO. 2 SHED.

at each column when the two sheds were constructed in 1921 to the design of Sir Cyril Kirkpatrick, Past-President Inst. C.E., the Authority's Chief Engineer from 1913 to 1924.

During the demolition and removal of "M" jetty by blasting and

Fig. 11.



VICTORIA DOCK: SECTION THROUGH SOUTH QUAY AT NO. 1 SHED.

dredging, a temporary staging was erected across the water area towards No. 2 Shed, the foundation piles for the new shed were driven, and the quay commenced from the west end. This consisted of the two rows of cylinders and continuous sheet-piling shown in *Fig. 11.* The latter was driven first and temporarily supported by steel walings at water-level and tied

back by $1\frac{1}{2}$-inch diameter bars to the base of the shed foundations. A 16-inch raking pile was then driven on either side of the sheeting at the 24-foot centres of the shed-columns and the area behind the sheeting filled in. The front cylinders followed by the back cylinders were then sunk, piled, and hearted. The cylinders sunk in pits from a level of +7·00 N.D. at the extreme west end proved troublesome, and it was necessary to link the rings together. Pre-cast soffit-beams were used for both the coping and the longitudinal beams; the main transverse beams were pre-cast up to the underside of the deck-slab with rods left projecting for welding to the land anchor-ties, and the intermediate transverse beams were supported from the main beams and concreted to the underside of deck at the same time as the coping, intermediate longitudinal, and sheet-pile capping beams. All transverse beams were formed with tapered sides to form a 2-inch rebate for 3-inch thick pre-cast deck-slab soffits reinforced with high-tensile steel, these being suspended from travelling trusses during the depositing and setting of the deck-slab.

The adoption of pre-cast soffit-beams not only overcame the difficulty of erecting and striking shuttering near the water-level, but prevented the pinning down of laden barges beneath the deep coping beams. The ballast-level fell from 8 feet 6 inches below N.D. at the west end to 16 feet below N.D. at the east end. The sheet-piles ranged from 28 feet to 38 feet long and the front-cylinder piles from 40 feet to 50 feet long. A 5-inch fresh-water main was hung under the quay with hydrants placed on cylinder-tops at 96-foot centres. Access manholes were formed in the deck at 216-foot centres and two pitch-pine rafts were provided and moored under the decking for the maintenance of the main.

A 13-foot 6-inch gauge crane-track was laid the full length of the quay with slots, 2 inches wide and 3 inches deep, across the quay at about 60-foot intervals to take the trailing cables of the fifteen 3-ton electric cranes to plug-in boxes on the electric mains fixed on the shed sides.

Two railway tracks with crossovers at the shed openings were provided, also five 2-ton electric capstans and fairleads.

In this and in other reinforced-concrete false-quays described later, thick deck-slabs have been adopted to obviate waybeams, so that all crane- and railway-tracks and crossovers may be laid in any position desired on the quay, and also to take the wheel-loads of the heaviest road vehicles.

The sill and aprons of the Tidal Basin passage were broken up by a 10-ton ram rock-breaker and removed by bucket-dredger, which also deepened the berths and south side of the dock to 31 feet below impounded-water level.

The foundations of the new shed generally consisted of groups of four 16-inch square piles up to 35 feet long driven to a set in the ballast, which averaged −10·00 N.D.; the pile-caps were 5 feet square and 3 feet deep, tied together transversely at 24-foot centres and longitudinally at the sides and centre by 12-inch by 21-inch tie-beams all formed of 1 : 2 : 4 concrete.

The shed is 504 feet long by 120 feet wide, built of reinforced-concrete framework and brick panels, and its three floors, each with a clear height of 12 feet, are completely divided into three sections by fire-walls. The upper floors are $6\frac{1}{2}$ inches thick and are designed to carry 3 cwt. per square foot. The roof is 6 inches thick covered by three-ply "Ruberoid" sheeting with 2 inches of crushed gravel laid evenly over the surface; it is pierced by twenty-four 18-foot by 7-foot clear openings for lantern-lights fitted with louvre type ventilators and glazed with $\frac{1}{4}$-inch non-actinic glass to reduce heat-transmission. A 2-ton electric lift is provided at each division wall with two fire doors to each floor. Three 1-ton electric luffing cranes with a radius of 20 feet are provided on the road side and one at each gable. The 8-foot wide platform on the road side is roofed over, except at the crane bays, and the railways are paved for vehicular traffic.

These works were carried out by Messrs. J. Mowlem & Co., Ltd., and were completed in 85 weeks. The total cost was £270,000, the main items of which are: new quay, £70,000; shed and foundations, £91,000; plant and electric mains, £60,000; and £31,000 for the removal of "M" jetty (Fig. 1, Plate 1) and deepening the berths to 31 feet below impounded-water level.

The Mudfield Site.

This area of 27 acres at the south-east corner of the Royal Victoria dock has remained undeveloped due to the dumping thereon—possibly when the Pontoon dock was formed—of a large quantity of excavation which covers about two-thirds of the area to a height of 15 feet above impounded-water level.

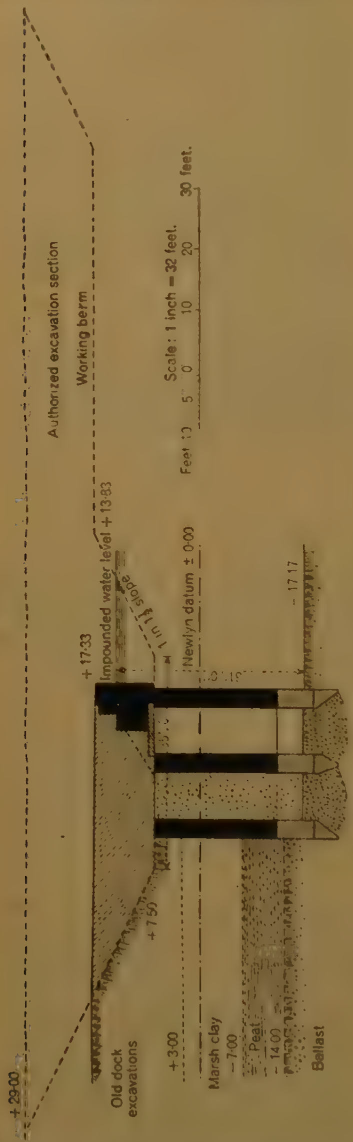
The first stage in the development of this site was commenced in August, 1936, when the tender of Messrs. Sir Robert McAlpine & Sons, Ltd., was accepted for the construction of a main quay about 1,200 feet in length on the same alignment as the existing south side quays, and of a return quay 560 feet long towards the Pontoon dock, with railway connexions and new road approaches.

The main quay consists of a mass-concrete quay-wall (*Fig. 12*, p. 302) founded on forty-two concrete monoliths, and the west quay of a modified wall and reinforced-concrete tunnel (*Fig. 13*, p. 303) founded on eighteen concrete monoliths.

The excavation to the specified monolith sinking-level of +7·50 N.D., or 6·50 feet below dock-level was taken out to "authorized sections" (see *Fig. 12*) by diesel drag-line excavators, and after allowing for filling behind the quays the surplus was deposited on the dock side of the construction tracks, to be ultimately removed by the dredgers. Attempts to fill in the dock water-area at the north-west corner with this material and to form a bank from which to sink six monoliths were unsuccessful as the material turned into slurry when tipped, and it was found necessary to deposit about

40,000 cubic yards of dredged ballast before a stable bank was obtained
Most of the ballast was afterwards redredged and used to form land on th
north side of the Victoria dock.

Fig. 12.



MUDFIELD SITE: SECTION THROUGH MAIN QUAY.

The monoliths were 24 fee
6 inches square with four 7-foo
9-inch wells and were 30 feet
inches deep, when sunk, belov
the sinking level. The shoe
were cast to a height of 10 feet
the upper 6 feet of 6 : 1 ballas
concrete and the lower 4 fee
of 1 : 2 : 4 concrete, reinforce
with eight 1¼-inch diameter bar
in each side and seven 1⅛-inc
diameter bars in each cross
wall.

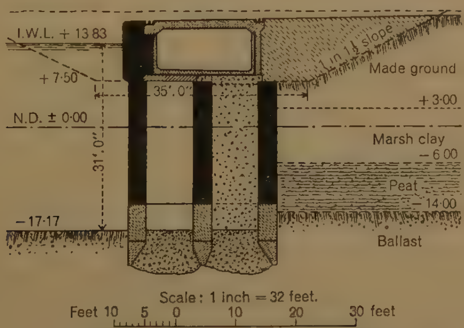
The cutting edges were 6 inche
wide with a taper of 2 feet
inches in a height of 3 feet 6 inche
(Figs. 14, 15, and 16, Plate 1)
After sinking 9 feet by grabbin
and loading, strong pitch-pin
shutters were erected, strutte
and bolted together for the secon
cast of about 10 feet, which wa
sunk 9 feet as before; the thir
cast followed and the sinkin
stopped at 23 feet below N.I
The amount of kentledge re
quired for loading varied con
siderably, being up to 800 ton
through peat and 540 ton
through ballast. For this pur
pose 4- and 5-ton blocks of cast
iron were used.

The monoliths were pitche
12 inches behind the cope-lin
and 4 feet 4 inches apart; th
worst final position was 7 inche
forward and 18 inches sideway
of the pitched position. The onl
troubles experienced in sinkin
were at the north-west corner where the monoliths were sunk from 16 fe
above N.D., through the tipped ballast bank, and struck the hardwoo
frames of an old ship which probably had been scuttled, and also not fa

f this, where a large wood-framed pickling tank was encountered full of a atted mass of creosoted wood rubbish. After pumping the wells the ardwood was blasted and drawn, and the creosoted rubbish dislodged by horizontal water jet from a 6-inch vertical electrical pump lowered down undercut it. *Fig. 17* (p. 304) shows the rate of casting and sinking of e monoliths of each quay.

After standing loaded for 24 hours, the four wells of each monolith ere cleaned by divers and sealed with 5 : 1 concrete deposited from self-scharging skips to a height of 6 feet above the cutting edge. The back ells were then filled to within 6 inches of the surface with 12 : 1 hearting oncrete. All four wells of the main corner monolith were hearted. The aces between the monoliths were sealed at the back by driving into the allast four 15-inch octagonal concrete piles arranged in arch form, the eads of these 30-foot-long piles were stripped, and the bars bent over into

Fig. 13.



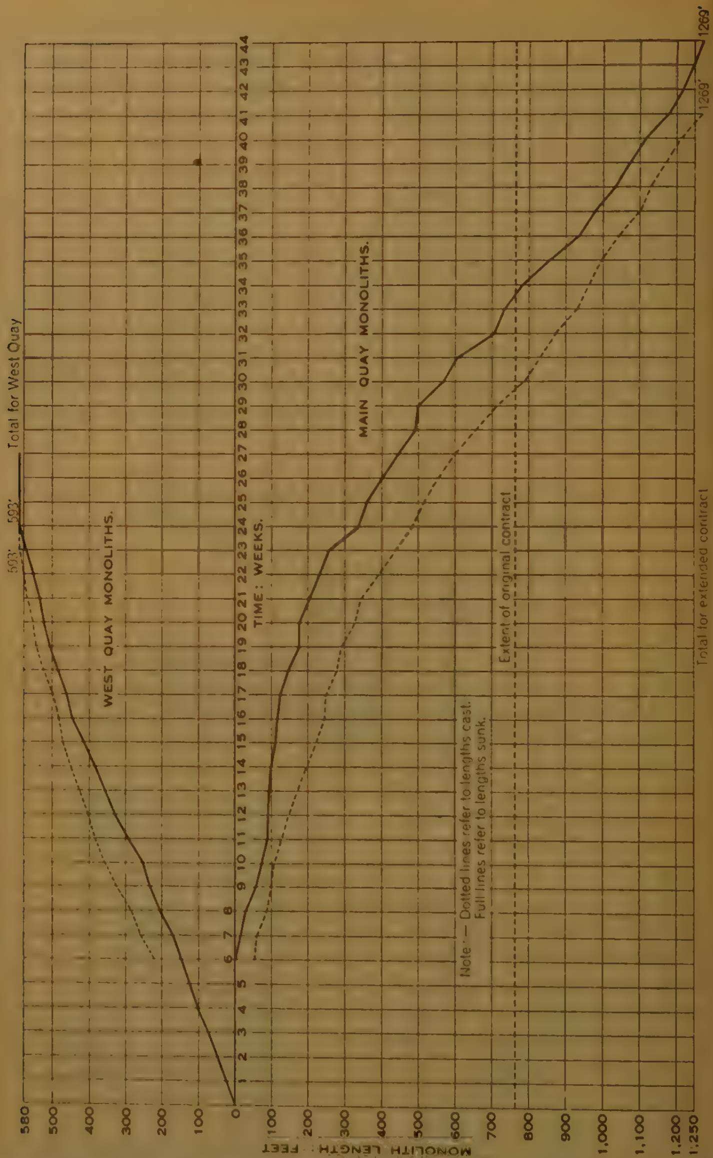
Mudfield Site: Section Through West Quay.

he concrete slab over the spaces. The front wells were covered with 2-inch pre-cast reinforced-concrete slabs.

At the south-west corner, the quay-wall of the Pontoon dock (the oundations of which were only 15 feet below dock-level) was enclosed ithin a steel-piled box-dam, cut down for a length of 30 feet and rebuilt n concrete from the same foundation-level as the monoliths. The space etween the new wall and the last monolith was closed with octagonal piles nd covered as described above.

A grain-conveyor tunnel 15 feet 6 inches wide with an average height f 7 feet was formed in the topping wall of the west quay (Fig. 18, Plate 1). 'or this purpose the tops of the monoliths were made up to a uniform evel in 6 : 1 concrete and finished to a reasonably smooth surface for aterproofing. After the construction of the quay-wall and 12-inch thick einforced-concrete back wall a 6-inch strip of "Ruberoid" was laid long the bottom angles in three layers. The walls were treated in the

Fig. 17.



Mudfield Site: Rates of Casting and Sinking Monoliths

same way and all side and end joints were staggered. Immediate after laying the waterproofing courses a supporting lining of 1 : 2 : concrete was placed on the floor and walls, and this was followed b the casting of the 16-inch thick reinforced-concrete roof pierced at abou 18-foot intervals with frames and covers for grain-spouts. Asbestos-cemen vent-pipes, 4 inches in diameter, were placed under the coping betwe

the tunnel and dock faces. An elevator-track, consisting of two pairs of 75-lb.-per-yard flat-bottom rails with centre flange-way at 18-foot centres, was laid along the quay, and the quay surface graded at 1 in 60 to the dock.

The topping wall of the main quay, also constructed to a cope-level of +17·33 N.D., was cast in 60-foot lengths in two lifts, the front shuttering being in 24-foot lengths with a full depth of 10 feet. In addition to tarring the keys of the vertical construction-joints, two 1-inch expansion-joints of bitumastic sheeting were provided in the total length. Cope bollards are provided at 85-foot centres, anchored by tie-rods in counterforts into the back well of the monoliths.

During the construction six 15-ton steam travelling-derricks with a jib-radius of 80 feet were in use, grabbing, moving kentledge, shutters, and concreting. The materials for concrete were delivered by water to bunkers and two 1-cubic-yard electrically driven mixers erected on the north foreshore, and the concrete was conveyed to the derricks in skips on bogies hauled by 3-foot gauge locomotives.

Prior to the construction of a new road-approach from the highway to this area of the dock an access road 20 feet wide of tar-macadam on hardcore was formed from Mill Road gate round the south side of the Pontoon dock to the site. Railway sidings and a junction with the Silvertown line have been provided, and by an exchange of land with the West Ham Corporation, and the construction of a retaining wall, a railway connexion to the dock lines westward was made across a former portion of the recreation ground.

These works were completed in 55 weeks at an inclusive cost of £175,000. The quays cost £124,000, which is at the rate of £70 per foot run.

A quantity of the material in front of the main quay is required for the new work on the north side of the Victoria dock and is being removed under the dredging contract for that work. When the dredging at this site is completed to 31 feet below impounded-water level, the deep water width of the "canal cutting" will have been increased from 250 feet to 600 feet. The width of the entrance to the Pontoon dock was increased from 125 feet to 340 feet to allow a passage for craft between vessels moored at the quays, each with floating grain-elevators delivering overside into craft.

Remodelling North Side of Victoria Dock.

Having completed the deepening of the Connaught Road passage and provided some alternative berths and shed accommodation on the south side, the Port Authority decided to proceed at once with the remodelling of the north side of the Royal Victoria dock, where nearly one-half of the total quayage was non-effective and only about one-tenth of the effective portion was modern in character, and it was possible to obtain accommodation for vessels up to 29 feet draught at modern quays without the initial heavy cost of a new entrance.

Between berths " A " and " Z " at the east and west ends reconstructed and equipped for the discharge of meat, the existing accommodation consisted of five solid jetties constructed with horizontal arch panels of 14-inch brickwork between cast-iron columns built up from the dock bottom at 7-foot centres and held in place by two tiers of 2½-inch diameter tie-rods upon which the stability of the quay-walls depended. These tie-rods were inaccessible for inspection and their condition not known until revealed by a collapse or bulging. There were also three intermediate shorter timber jetties generally used by discharged vessels. All these jetties were inadequate to deal at one time with the cargoes of vessels lying on each side, and thus only one-half of the quayage was effective; moreover the structures were too frail to accommodate modern cranes, and too short for the vessels requiring to use them. These facts are not surprising when it is realized that the jetties were built 80 years ago, and that the whole dock is reputed to have cost only £730,000. It was the first dock in the port to have railway facilities, and also the first to be equipped with hydraulic power for operating the machinery.

The scheme adopted consists of a continuous open-type reinforced-concrete quay about 3,250 feet long, sited to provide width for a 150-foot shed, railways, and road, all clear of " X " warehouse at the west end and lining up with " A " berth at the east end where the minimum width of the water area is 600 feet. This involved the removal of the southern end of the jetties and the reclamation of the water area on which five modern three-storey warehouses, each 504 feet by 150 feet, are to be erected and provided with road and rail facilities; the rail access to the two tracks on the new quay being effected by an opening 240 feet wide between the sheds near the centre of the quay. The estimated cost of this work, including crane equipment, is £1,200,000.

Contracts amounting to £341,500 have been entered into with Messrs. J. Mowlem & Co., Ltd., for the construction of the quay in 26 months and with the Tilbury Contracting & Dredging Co., Ltd., for the removal of the jetties, deepening and land reclamation where required. Work was commenced at each end of the site in January, 1937.

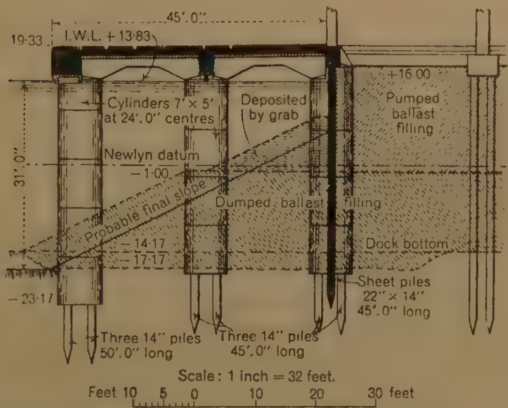
At the west end cylinders were sunk and the small recess decked over for the extension of " Z " shed and quay 120 feet eastward to " I " jetty, where pre-cast sheeting piles were driven along the west side of the jetty prior to forming a false quay on one row of cylinders. At the same time the demolition of " I " jetty by blasting, cutting tie-rods, and dredging was proceeding, the dredged material being deposited from small hoppers in the shore ends of the bays after the removal of the small timber jetties and the accumulated mounds of mud underneath them. These timber jetties each contained from three hundred to four hundred piles from 40 to 45 feet long, which were extracted by a wreck lighter at an average rate of thirteen per day, the highest number per day being twenty-four.

The masonry of the gate-recess and remainder of the sill at the south

nd of "I" jetty were broken up by a 15-ton rock-breaker and removed by dredger.

The procedure followed for the construction of the new quay (*Fig. 19*), consisting of three rows of cylinders with strong transverse beams support-

Fig. 19.



VICTORIA DOCK: SECTION THROUGH NORTH QUAY.

ng 14-inch thick concrete sheet-piling for retaining the filling of the areas o be reclaimed, was as follows:

The mud was removed from the dock-bottom, and ballast from the leepening of the dock was deposited from small hoppers up to a level of −1·0 N.D. thus leaving 15 feet of water over a top width of 200 feet equired for the quay and shed. A temporary staging of five rows of imber piles 45 feet long at 24-foot centres was then erected, for supporting he steel guide-cages and to carry the sheet-pile frame and a 5-ton steam rane near the back row of cylinders. A 7-ton floating crane was used for inking and hearting the front cylinders (Fig. 20, Plate 1), pitching piles, nd placing the pre-cast soffit-beams in position. A floating mixer-plant vas used for the hearting, cope and back beams and a concrete pump for he rest of the beams and deck-slabs.

The ballast bank was stopped about 200 feet short of the next jetty o be demolished, that being the length required for the dredger to work hrough. The removal of each jetty end took about 5 months, as about a nonth was required for each of the following operations:—

(*a*) Demolition of the southern half of brick shed.
(*b*) Demolition of arches to vault floor-level.
(*c*) Drilling and blasting shed foundations.
(*d*) Drilling and blasting quay-walls.
(*e*) Dredging and removal of jetty end.

The jetties were removed to 68 feet behind the new cope-line, an before depositing the ballast bank the dredger made a searching cut t −17·0 N.D. over the site of the cylinders, which was also raked over b three grapnels attached to a heavy drag towed by a small tug.

On completion of the new quay between two of the solid jetties the wate area was reclaimed to +16·0 N.D. (that is, to the level of the undersid of the shed foundations), by pumping ballast through a 24-inch pipe-lin from pumping plant moored alongside the new quay, the ballast bein obtained from the deepening of the dock and conveyed in special barge Northward of the new shed sites, demolition materials were deposited u to existing quay-level as reserve filling.

Progress at the west end was delayed about 6 weeks owing to the per colation of dock water from the vaults of "G" jetty, which are about 7 fee below impounded-water level, into the lower basements of a large warehous on the north side of the roadway. It was found that the wate was travelling through the lime-mortar joints of the brickwork foundation of the vaults, and thence along the outside skin of a tunnel connecting them to the warehouse basement. The trouble was not cured until interlockin steel sheet-piling had been driven across the tunnel and half the width o the jetty.

It should be mentioned that the roof-water of the jetty warehouse was run into the vaults and they were drained through brick barrels lai at marsh-level into the marsh-ditch along the dock boundary; before an jetty was flooded a trench was cut across the north end of the vaults an all outlets plugged and sealed under a head of concrete.

Expansion-joints 1¼ inch wide were formed about 336 feet apa along the quay by 12-inch wide trimmer beams on steel bearing plates. I may be of interest to record the centres at which the following qua equipment was provided:

Cope bollards	96 feet
Life ladders	192 ,,
Barge-mooring-rings	48 ,,
Fresh-water hydrants	96 ,,
Access manholes	192 ,,

The standard radius for railway lines to the road side of the sheds 7 chains, but for access to the quay lines the radius is slightly less tha 4 chains.

General.

The deepening of the north quay and channel of the Royal Albe dock to 34 feet below impounded-water level was completed in 1936, an when the deepening of the Royal Victoria dock to 31 feet below impounde water level is accomplished, the total increases over the original depths these docks will be 7 feet and 5½ feet respectively.

Since the completion of the deep-water river-channel in the spring of 21, a depth of 30 feet below low water spring tides has been available om the Nore to Coldharbour Point, and 27 feet below the same datum up the Albert dock.

For many years the period of locking for shipping was $3\frac{1}{2}$ hours before, $\frac{1}{2}$ hour after, high-water, but in 1926 this was increased to 5 hours before

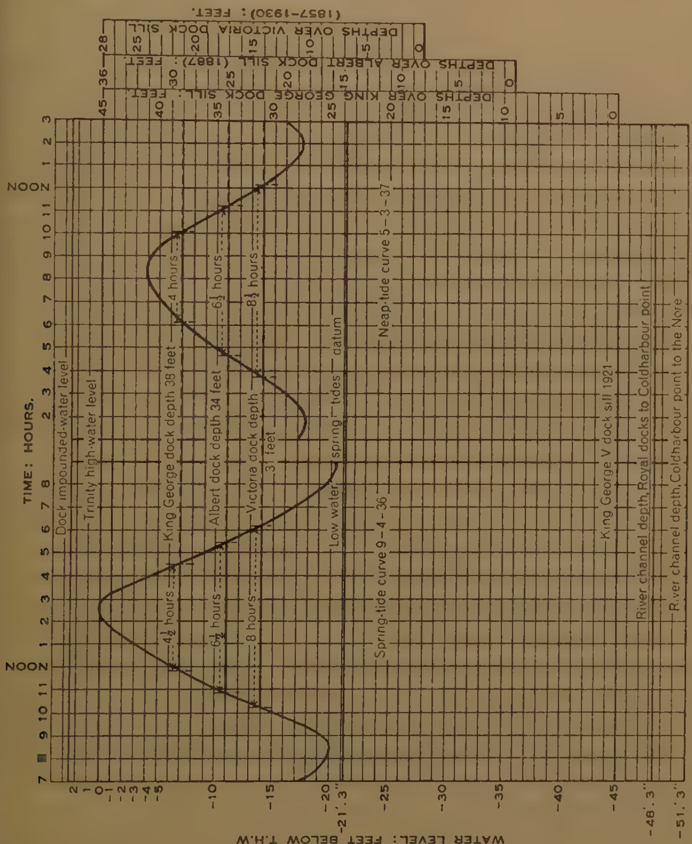


Fig. 21.

DEPTH OF WATER AVAILABLE OVER THE SILLS OF KING GEORGE V DOCK ENTRANCE.

3 hours after. This 8-hour locking is still worked at the King George V itrance, but in 1932 it was altered at the Albert dock entrance to 4 hours ch side of high water for shipping and craft.

The accompanying diagram (*Fig. 21*) shows the period of time the depth water in the three docks is available over the sills of the King George V itrance, and indicates that the shipowners' ideal conditions of docking any state of the tide, even for a 31-foot dock, would require a lock- and ver-depth of not less than 31 feet at low water spring tides.

For convenience, the relationship of the data referred to is as follow for the Royal Docks :

Newlyn datum (N.D.) is 1·17 feet above Ordnance datum.
Trinity High Water is 12·5 feet above Ordnance datum.
Impounded-water level is 15·0 feet above Ordnance datum.
Low water spring tides (L.W.S.T.) is 8·75 feet below Ordnance datun

Acknowledgements.

The recent improvements in the Royal Docks described in this Pape authorized by the Port of London Authority under the Chairmanship c Lord Ritchie of Dundee, were carried out to the design and under th direction of Mr. Asa Binns, M. Inst. C.E., Chief Engineer, and Mr. F. W. I Davis, M.Sc., M. Inst. C.E., Chief Assistant Engineer. The Author ha supervised the works at the Royal Docks since 1930, assisted at variou times by Messrs. A. F. Grant, B.Sc. (Eng.) and John Anderson, Assoc. M Inst. C.E., Mr. S. T. Smith, Mr. A. L. Wheeler, B.Sc. (Eng.), Stud. Ins C.E., and Messrs. W. T. Oliver and W. H. Ashmore, senior Clerks of Work

The Author would like to take the opportunity of mentioning th happy association with his colleague Mr. R. E. Knight, M.C., M. Inst. C.E the mechanical Assistant Engineer at the Royal Docks, and with Mr. D. (Davies, who supervised the mechanical and electrical equipment respect ively of the new works described.

The whole of the contract drawings were prepared in the Po Authority's drawing office under the supervision of the Chief Draughts man, Mr. W. B. Hall, M. Inst. C.E.

The contracts were based on firm schedules of prices both for labou and materials and Messrs. George Corderoy & Co. acted throughout as th Quantity Surveyors.

The main Contractors were represented on the sites by Mr. J. Hardi and Mr. E. Downes, M.C., B.Sc. (Eng.), Assoc. M. Inst. C.E., for Sir Robe McAlpine & Sons, Ltd. ; Mr. R. Swager, Assoc. Inst. C.E., and Messr J. A. Ross, Assoc. M. Inst. C.E., and B. S. Holmes for Messrs. J. Mowle & Co., Ltd. ; and Mr. J. H. W. Turner, B.Sc., Assoc. M. Inst. C.E., fo Messrs. C. Brand & Son, Ltd.

The Paper is accompanied by twenty-three sheets of drawings, fro some of which Plate 1 and the Figures in the text have been prepare and by seven photographs.

Discussion.

The Author, in introducing his Paper, showed a number of lantern-slides illustrating the progress of the work and the methods adopted in carrying it out.

The Chairman, in proposing a vote of thanks to the Author for his Paper, remarked that it contained a very valuable record of historical work, and also a great wealth of detail. Records of that kind relating to work which had been done so recently were of great value.

Mr. Asa Binns remarked that the works might be generally described as improvements rather than as extensions. The docks of the Port of London Authority dated back to the end of the eighteenth century, and it was not surprising that of the £20,000,000 spent in capital improvements since 1909, a substantial proportion had been spent in improvements to the old docks calculated to fit them for meeting modern traffic requirements, rather than upon extensions or the development of new sites. When the extraordinary growth in the size of shipping was considered, it was perhaps surprising that the layout of the old docks was such that they could be improved to deal with modern needs.

The provision of a false quay at the Royal Albert dock, with a sloping bottom dredged out to the extra depth necessary, was quite a common form of construction. It was cheap and efficient, and excellent so long as the wall was on a foundation which would stand up to the slope of the new bottom. There was no doubt about that in the case in question because there was a ballast bottom. One objectionable feature of the scheme was that it narrowed the dock by 20 feet; the Albert dock was very busy and at times there was congestion of water-borne traffic, so that if ever the south quay were dealt with in the same way the effect would be still more objectionable in increasing that congestion.

The work could have been carried out in easy stages, by starting at one end, with the minimum of disturbance. Various alternatives had been considered, but ultimately it had been decided to carry out the work very quickly, working from both ends; that had caused quite a considerable outcry from the shipping interests at times.

When the work of deepening the Connaught Road passage had been started, it had been hoped that divers would be able to peel off the brick courses one after another with pneumatic hammers and drills. That, however, had proved to be impossible, and the mortar, being quite as strong as the brickwork, came up in fragments, which made the work all the more difficult in view of the danger of a break-through. A break-through would have meant the flooding of the tunnels, of Silvertown, and

of East Ham, and the putting aground of the whole of the shipping in al three of the Royal docks, and the greatest care was necessary in carrying ou the work in order to avoid such a catastrophe. With the diving bel reliance had not been placed merely on lowering the bell through the 3(feet of water ; it was connected to the air-supply at the dry dock, and they blew the bell out so that the men who were working were only ankle-dee in water when cutting out the brickwork.

A few words might be said about the Victoria dock, which had given a lot of trouble and was most unsatisfactory. The entrance lock had collapsed before the dock was opened, the quays had never been fit for carrying the traffic—at least for the 30 years with which he had been associated with the dock—and in various sections they had collapsed and had been stiffened with sheet-piling. A word might be said about the curious Pontoon dock, which had not been constructed by the Victoria Dock Company but by a graving-dock company, who intended to repai vessels in the narrow bays provided. The vessels were lifted by hydraulic rams, put on to pontoons, and taken into the bays for repair.[1]

After the Silvertown explosion, Sir Cyril Kirkpatrick, Past-President Inst. C.E., had laid down roads and railways and had built two first-rate sheds, Nos. 2 and 3. Later on, when the Ministry of Transport had carried out their scheme for the Silvertown Way, which had so improved access to the whole area, the Port Authority had agreed to the bridge crossing the western entrance being a fixed bridge at a sufficient height to clear barge traffic. After that had been agreed upon they had taken in hand the complete repair of that entrance lock by putting a new invert in it in the dry, with a sufficient depth to take barge traffic ; all shipping using the whole of the three docks had now to be locked at the Gallions entrance, where there were three excellent locks.

He would like to put in a general plea for the improvement of shipping berths. It was astonishing to find magnificent ships needing deep-water approaches, expensive dredging and entrance-locks, quay-walls, and so on, being dealt with at berths with uneven paving, inadequate cranes, bad lighting, and bad ventilation. It was no wonder that there was carelessness in the handling of goods.

So far as the wall in monolith construction was concerned, there was the usual trouble with obstructions. The monoliths were originally designed to be constructed in block-work in 2-foot 6-inch courses. The contractors asked to be allowed to build them in mass-work, and after making inquiries and discussing the matter, it was agreed that they should be built in mass-work, and he believed that it had made a better job. The north quay was not yet completed. It would make a magnificent quay with five berths, and Mr. W. P. Shepherd Barron, M. Inst. C.E., his successor as Chief Engineer of the Port of London Authority, had already

[1] E. Clark, "The Hydraulic Lift Graving Dock." Minutes of Proceedings Inst. C.E., vol. xxv (1865–66), p. 292.

placed a contract for the five new sheds, which would be similar to No. 1 shed but wider and longer. No doubt later on Mr. Shepherd Barron would equip the Mudfield quays equally well, and when the whole of the works were completed the group of docks, with their wonderful situation so close to the City of London and with the deep water available, and with the fine road and railway access, would continue to maintain their supremacy, at any rate in the Port of London, for the handling of ships' cargoes.

In conclusion, he would like to say a word of thanks to the engineering staff, whose constant care was necessary in the carrying out of the works and in their design. All concerned had worked most loyally, and he appreciated their services most highly. The Port of London Authority were privileged in being able to secure the services of public-works contractors who had had long experience in carrying out works of the character in question. and for such works it was necessary to acquire a technique which nothing but experience could give; he would like to pay a special tribute to the professional and working staffs of those contractors.

Sir Cyril Kirkpatrick, Past-President, said that the work which had been described was really the 1910 scheme of the late Sir Frederick Palmer, Past-President Inst. C.E., with important additions and amplifications. One outstanding alteration and addition was the new tunnels inserted in the old brick tunnels, and the deepening of the Connaught Road passage. That was not in Sir Frederick Palmer's scheme, and during his own time as Chief Engineer it had not been seriously contemplated. He believed that he was right in giving to Mr. Binns the sole credit for taking that step and for advising that the tunnel should be strengthened and the passage deepened.

The two railway approaches, north and south, were at marsh-level. Mr. Binns had referred to the possibility of flooding Silvertown; he had had some experience of flooding when the King George V dock was being constructed, because a "blow" had occurred on the south side. They had previously had to make a long ditch for the Commissioners, and the blow just equalled the capacity of the ditch, so that they were able in every tide to get rid of all the water which collected. That was a fortunate circumstance, but the incident gave him an insight into what might happen, and he wondered what would occur if it had increased and it had not been found possible to stop it.

It was stated in the Paper that the deepening of the passage at Connaught Road had cost £70,000. Sir Frederick Palmer's scheme had been to make a new entrance at the Victoria dock, which would probably have cost 10 times that sum. He thought it would be agreed that the method which had been adopted of dealing with all the traffic from the down-river end instead of any shipping coming in at the west end, was much more satisfactory.

The Author referred to the pumping station at the east end of the docks. When that had been under consideration, Sir Cyril had asked Mr. C. E. Lefroy, Assoc. M. Inst. C.E., to carry out some experiments and tests to

find out what was really the most economical time for pumping, having regard to the state of the tide and the amount of silt in the water. From the notes made at the time, it appeared that the average silt-content to a depth of 15 feet at 2 hours before high water was 0·46 gramme per litre, whilst at 2 hours after high water it was 0·09 gramme per litre. That showed fairly clearly that if the pumping could be done on the ebb-tide it would be much more economical from the point of view of dredging than if it were done on the flood tide, but it was found that to do so would involve so much increased cost for electricity that the idea had to be abandoned and an equal time of 3 hours on each side of high water was adopted as being the most economical in the circumstances.

Sir Henry Japp congratulated Mr. Asa Binns on his adoption of the false-quay system, which enabled the dock-wall to be preserved while giving deep enough water for modern ships. The Port of London Authority, by adopting that scheme, had been enabled to modernize the Royal docks and to improve them so greatly that it looked as though the big dock, which all contractors had been hoping for, on the north side of the Albert dock, would not be the subject of tenders for a long time. It was said that the need for protective tariffs had so reduced the mercantile marine that it would be many years before that dock was built.

The first time that he had had experience of a false quay was when he had carried out some work on modernizing the old Lonsdale dock at Workington to the design of the late Sir Frederick Palmer. Sir Frederick had made those false quays with reinforced-concrete piles and very expensive horizontal and diagonal bracing. Mr. Binns, by adopting cylinders to stiffen the piles, had been able to avoid that expensive work of bracing and had secured an equally good result. Sir Henry's first experience of the use of reinforced-concrete cylinders with piles in that way was in connexion with the work of the late Mr. H. A. Reed on the Manchester ship-canal, where instead of using reinforced-concrete piles very strong steel cruciform piles were driven inside the cylinder and a longitudinal pre-cast girder weighing 60 tons was supported on the cylinders, the girder being hollow and forming the tunnel for a grain-conveyor.

The reinforced-concrete girders used in the works described by the Author were first of all built on the soffit forms, and after the lapse of the required time the shutters were removed, because it was impossible to get to them after the decking was built; the Author had therefore provided for trusses to be placed across the top of them and the weight of the girders to be carried by bolts from the trusses. The girders at that time contained the bottom tension-bars, but none of the steel bars at the top for compression, as the girders were ultimately T-girders, the T part being in the decking. The nuts on the bolts were not tightened to any determined amount, and it therefore would be quite possible to lift the girders and to have them bend upwards to some slight extent, so that when the decking was fixed and the bolts were ultimately released there would be a greater deflexion than would ordinarily occur; in fact, they were

bound to deflect until the steel came into play. That might result in more fractures of the soffits of the beams than was usual, but it would be impossible to see that because no one could get under the decking unless the water in the dock was lowered. A good deal had been heard about pre-stressing steel for use in reinforced concrete, but the question which he was discussing seemed to be rather one of de-stressing it.

It was impossible to get underneath the quay-wall in the Victoria dock after the front beam was built, because the water lapped the under side of it; the Author had used pre-cast girders for the longitudinal girders, and by putting rebates in the cross-girders he had been able to put a light pre-cast slab of reinforced concrete, 4 inches thick, to act as a shutter to carry the slab of the deck. By designing the work in that manner it was possible to get it done cheaply and quickly.

Mr. James Conacher remarked that, as a member of the staff of one of the contractors, he would like to make a few remarks about the Royal Albert dock. He realized that a contractor was going to be a great nuisance to those conducting the working of the dock traffic, because the docks were very busy and it meant changing regular liners from their berths, and from the sheds adapted for their use. There was nothing else which could be done but to work for a speedy completion, and it had been possible to co-operate perfectly with Mr. Binns and with the Author to secure that end. Two plants had been employed, working from each end towards the middle, and they were able to complete as much as 360 feet of quay per week in the later stages of the work.

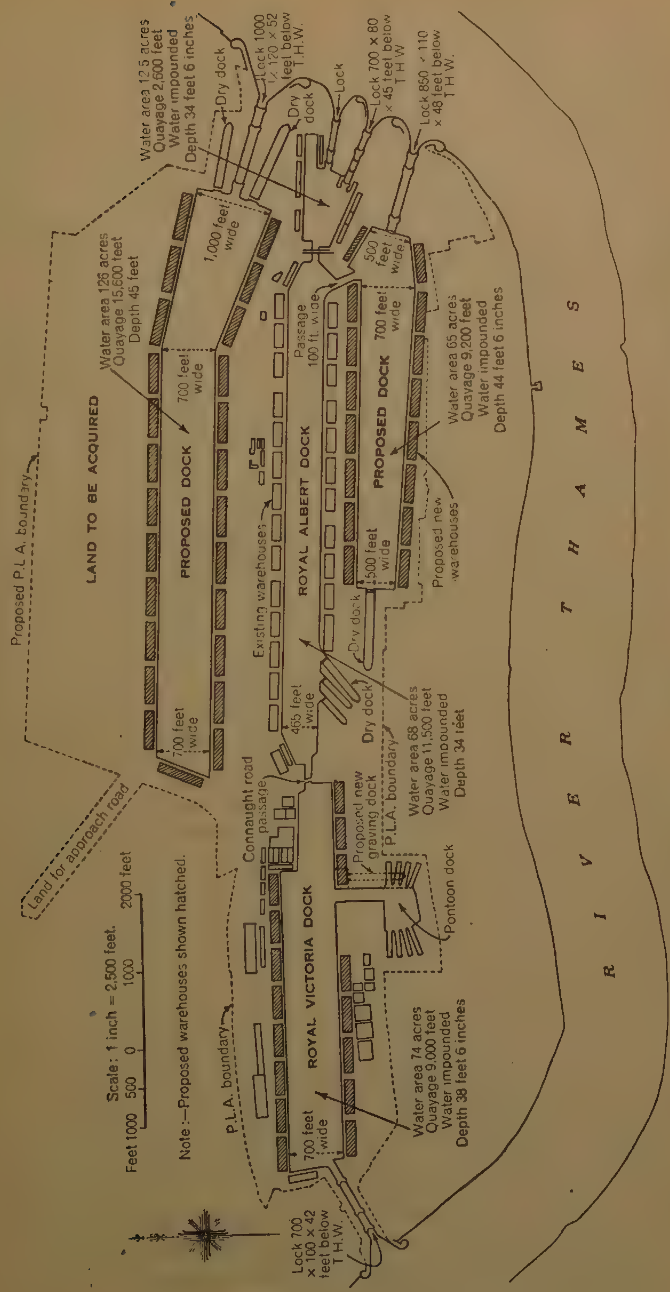
The works on the Mudfield site comprised an ordinary monolith quay-wall. The monoliths were of excellent design, and the excavation was mostly in marsh clay "bungum," very much stiffer than the "bungum" which had been met with at Tilbury. At Tilbury the material used to flow to the grab, but at the Mudfield site that did not occur, and that was reflected in the amount of kentledge which had to be used on the Mudfield site as compared with the works at Tilbury. The monoliths at Tilbury, which were 25 feet square, could be put down with a maximum of about 400 tons of kentledge, whereas for the monoliths at the Mudfield site almost 800 tons were required. Curiously enough, when the gravel underlying the "bungum" at Mudfield was reached, it was possible to work with less kentledge. It was not necessary to de-water the monoliths at the Mudfield site, and in any case, since "bungum" was being dealt with, he did not think it would have been much use to do so, because the object of de-watering was to induce a flow round the base of the monolith so as to reduce skin-friction. It was rather difficult to estimate what the skin-friction on the monoliths was, but as nearly as could be estimated at Mudfield it was between 8 and 9 cwt. per square foot.

Dr. Brysson Cunningham thanked the Author for giving such very full particulars of costs; in that connexion he would like to congratulate Mr. Binns on having carried out the deepening of the north-side quay in the Albert dock at such a remarkably low rate as £15·6 per linear foot.

Dr. Cunningham thought that the method employed was perhaps as cheap as any that could have been adopted, whilst it produced an excellent arrangement.

Sir Cyril Kirkpatrick and others had alluded to the original re-modelling scheme for the Royal Docks, put forward in 1910 by the late Sir Frederick Palmer while Chief Engineer to the Port of London Authority. Having held at that time the position of Personal Assistant to Mr. Palmer (as he then was), and therefore having taken part in the preparation of his scheme, which was shown in *Fig. 22*, it might be permissible for Dr. Cunningham to point out the more notable differences between it and the scheme actually carried out. On the north side of the Royal Victoria dock, Sir Frederick had decided upon a straight quay, cutting right across the then existing jetties, but his line was not the same as that adopted by Mr. Binns, which lay farther south in continuation of the quay at " A " berth. On the south side of the dock, Sir Frederick Palmer's line came forward into the dock, leaving a distance between the two quays of 700 feet, whereas, in the scheme carried out by Mr. Binns, the distance was only about 600 feet, resulting in a reduction in width of about 100 feet, which, in view of the traffic in the dock, Dr. Cunningham thought insufficient. Sir Frederick Palmer had made provision for deep-draughted vessels in the dock, allowing a depth of 38 feet 6 inches, as compared with 31 feet, and had contemplated an entrance for them from the west, with a large lock 700 feet long by 100 feet wide, having a sill 42 feet below Trinity High Water. Mr. Binns had seen fit to abandon altogether the idea of a ship-entrance from the west, a step which did not commend itself to Dr. Cunningham, because vessels desirous of reaching berths at the extreme western end of the Royal Victoria dock had now to travel the whole distance of 2½ miles from the eastern entrance through a narrow and often crowded waterway, and that was bound to involve difficult manœuvring. A condition peculiar to the docks of the Port of London was the great amount of cargo delivered overside into barges, and that fact had to be borne in mind, since the barges often incommoded the passage of vessels. While during ordinary routine-operation the lack of an alternative entrance would prove inconvenient, the conditions in war-time had also to be considered. In the event of an air-raid warning it would be an extremely difficult matter to clear the docks of shipping through a single entrance, and he doubted whether it could be done in time. On the north side of the Royal Albert dock Sir Frederick's quay-extension width on each side was only 12 feet 6 inches instead of 20 feet, so that there would have been less restriction of the waterway than with two widths of 20 feet now contemplated. The present width of the dock was about 470 feet, and every foot was valuable. Sir Frederick had abandoned any idea of lowering the Connaught Road passage, and stated in his Report that he considered it to be impracticable ; that opinion had been endorsed by Sir Cyril Kirkpatrick, but Mr. Binns had succeeded in securing an additional 3 feet in depth, for which he deserved hearty congratulations.

Fig. 22.



THE LATE SIR FREDERICK PALMER'S SCHEME FOR REMODELLING THE ROYAL DOCKS (1910).

Mr. H. J. Deane remarked that reference had been made to the very difficult task which the maintenance of the Victoria Dock entrance had presented. There had been numerous attempts to stabilize the roller-paths for the lock-gates by divers, by packing up with brickwork and concrete ; one method which had been tried during the time that he had served with the Port of London Authority was very interesting. The great difficulty was to set on a bad foundation the cast-iron or cast-steel roller-paths and to maintain them more or less level to support a water-logged gate which weighed 95 tons. An attempt was made to overcome the difficulty by casting a trough section about 3 feet wide and 22 inches deep in reinforced concrete, and setting inside the trough the cast-steel paths, which were only temporarily bedded down pending the placing of the troughs in the bottom of the lock. After the chase had been cut out by divers, the troughs had been lowered into position and carefully levelled and packed up with concrete ; when the concrete had thoroughly set, the cast-steel roller-paths had then been placed in their proper position, and the entrance lock again set to work. It would be of interest to know the result of that method, and to know how long it had been really effective.

Sir Cyril Kirkpatrick had given some interesting figures with regard to the amount of silt, but had not suggested the great difficulty which arose in forming a proper estimate of the amount of pumping that the three large pumps at the Gallions locks would do. The length of quay was about 9 miles (excluding the King George V dock, which had not then been opened) and the total area of the docks was of the order of 8,000,000 square feet. The total newly-wetted surface was over 100,000 square feet and to get a real test of the amount of rise in such a large area required particular care. When it was realized that 1 minute's pumping on that large area represented only 0·07 inch difference in level, he suggested that those who were faced with that particular problem might consider the introduction of standing-wave flumes or venturi-meters.

The Author referred to the machining of the tunnel segments, and it would be interesting to know how that was carried out. He assumed that it was done in a planing machine, but he would like to know what the method was.

He fully appreciated the difficulties which were met with in the driving of the small pipe-tunnel, because in driving a number of tunnels in London clay similar difficulties had been encountered, and he had had a very great deal of trouble in doing some reclamation-work by means of a suction-cutter-dredger, when the only effective method for dealing with the septaria had been to drill holes and to crack them up by means of blasting.

Mr. F. M. G. Du-Plat-Taylor asked to what distance the deepening of the Albert dock was carried out from the quay ; was it carried half-way across the dock, or merely to a sufficient width to accommodate ships at the berth, and perhaps a vessel passing ? The south quay was not deepened.

The cylinders shown in Fig. 20, Plate 1, for the Victoria dock had three piles, and their dimensions were given, but the dimensions of the cylinders

at the Albert dock (*Fig. 4*, p. 290) were not given. They appeared to be circular and to have only two piles.

He had been interested in the work at Mudfield. The material there, he believed, had been obtained from the hydraulic lift dock about 1860, and it was not surprising that it was found unsuitable as filling. When he had been at the Victoria dock in 1905 it had been found to be quite unsuitable for carrying any load at all. An optimistic firm had rented the land with a view to making bricks out of the material, and they set up a good deal of plant and worked for about a year, but in the end they lost their money and left because the bricks broke up in the kiln; after that the site remained derelict until the scheme described in the Paper was carried out. He would be interested to know what was going to be done with the rest of the area. A large number of piles had been driven, apparently to carry a flour-mill, and presumably the rest of the site would be covered with sheds, roads, and railway lines, but in view of the nature of the material he imagined that everything would have to be built on piles, or else some kind of subsidence would take place such as took place at Tilbury docks, where the quays and sheds continually settled from the time that they were constructed.

He had been interested to hear that the skin-friction in the monoliths was almost the same as he had found it to be in the case of the first monoliths sunk at Tilbury dock. They were the first monoliths ever put down in the Port, and had been designed by Sir Frederick Palmer and sunk in the years 1912 to 1917. It had been stated earlier in the discussion that the material at Tilbury differed from that at the Mudfield site, but it seemed that the skin-friction in the two cases was about the same.

Mr. A. T. Best remarked that on p. 305 the Author referred to the change made in the layout of the north side of the Victoria dock, from a jetty or gridiron type to a straight quay. That alteration raised the general question of the life of engineering works. The Author remarked that the stability of the quay-wall depended on tie-rods, and that the jetties "were inadequate . . . too frail . . . and too short," and he added "These facts are not surprising when it is realized that the jetties were built 80 years ago, and that the whole dock is reputed to have cost only £730,000." The engineer of the Victoria Dock Company had been Mr. George Parker Bidder, Past- (then Vice-) President Inst. C.E., and it was one of his greatest works. Mr. Binns had commented on the dock being so close to the City of London, and it was true that transportation had since made it so, but at the time of construction it was regarded as a very venturesome and bold project to build docks right away "in the Essex marshes." However, the dock had been constructed there, and the jetties, which were one of the leading features of the design, came in for nothing but praise; in the discussion on a Paper[1] before The Institution Mr. Alfred Giles said "the plan of the jetties . . . gave the greatest facilities for the trade that

[1] Footnote (*), p. 284.

could be well devised." As recently as 1921 Sir Joseph Broodbank wrote [1]: "The value of the Victoria dock system of jetties is constantly attested by shipowners who are desirous of quick despatch in emptying their ships and delivering cargo." It was true that even in 1910 Sir Frederick Palmer had declared the dock to be "out of date," and had proposed a straight quay on the north side, but in the discussion on the Paper [2] referred to, Mr. Abernethy only criticized detail when he said he thought the system of construction adopted for the jetty-walls "did not possess that amount of security which was required." The iron tie-rods in a short time "would oxidize, and be apt to fail." Mr. Bidder, in his reply to the discussion, said, "In reply to the observations which had been made, as to the construction of the jetties, that they were not of a permanent character, referring no doubt to the wrought-iron tie-rods, it was desirable to know what was meant by permanence, whether it was thought they ought to last for fifty, a hundred, or five hundred years, as without some stated limit the observations were useless. It could hardly be asserted that, covered up as they were and excluded from the atmosphere, these tie-rods of 2 inches in diameter would decay and become useless within the next two hundred years." Not 200, but 80 years had since gone by, and it would be interesting if the Author could say what was the condition in which those tie-rods were found.

Mr. Asa Binns and **the Author** held up one of the tie-rods for the speaker's inspection, showing it to have become tapered to a point at both ends.

Mr. Best said that that raised another factor in the question; the life of engineering works might outlast their usefulness. Mr. Bidder had been a great mathematician, and perhaps he had reckoned it to be better that as an engineering work grew old and out of date in its general form it should simultaneously grow old and decay in construction; and possibly he reckoned that, if, instead of spending another £100,000 on the dock, he saved £100,000, in 80 years even at 3 per cent. that would become £1,000,000, and would be sufficient to pay for works such as those which had now been carried out.

Mr. D. C. Bean said that his remarks would refer particularly to the quay-wall at Mudfield. Some 3 years ago he had had the privilege of seeing the extensive reconstruction works carried out by the South Australian Harbours Board at Port Adelaide. The designing engineer had presented a Paper to the Institution of Engineers of Australia [3] in which he discussed the various possible wall-types in detail, and stated his reasons for the adoption of the type shown in *Fig. 15* of that Paper, as being low initial cost, small maintenance and the adaptability of the design to varying soil conditions along the length of the work. It had

[1] "History of the Port of London," p. 194. London, 1921.

[2] Footnote (*), p. 284.

[3] F. Andres, "Notes on the Development of Quay Walls." Journal Inst. Eng. Aust., vol. 7 (1935), p. 93.

occurred to Mr. Bean that that type of design might be equally applicable at Mudfield for the very same reasons. The chief features of the design were the reinforced-concrete superstructure, which was carried on a system of forward and backward raking piles with a curtain of sheeting in front. The stability of the wall was secured by the horizontal forces from the earth-pressure plus the ships' pull, combining with the vertical load of earth on the anchor-slab to produce an inclined resultant, which was transmitted through the anchor-slab to the system of raking piles. It had also the advantage of screening the sheeting from any extra earth-pressure due to excess loading. The wall had all the features desirable in dock-construction, particularly as it was very stiff in all directions; the anchor-slab acted as a very strong horizontal beam able to transmit the bump of a ship against it to a number of the pile trestles, and the front wall was stiffened by the counterforts to give again that same distribution on to other piles. Had that type of construction been considered for use at the Mudfield and, if so, why had it not been used? It had been extensively used on the Continent and in America for walls of even greater height than that described by the Author, and the cost had been stated to be only 75 per cent. of that of a monolith type of construction.

*** **Mr. C. J. S. Anderson** observed that the works necessitated by the deepening of the passage between the Albert and Victoria docks were of particular interest. While the alterations were being made to the railway tunnels under the passage, the London County Council was carrying out a somewhat similar project, but of smaller proportions, in strengthening the main sewer under Victoria street, Westminster. The old Victoria Street sewer was constructed about 1850, of egg-shaped section, in brickwork, to nominal sizes of 3 feet 4 inches wide by 5 feet high, and 4 feet by 5 feet 6 inches, its soffit being about 13 feet below street-level. Faulty work in the original construction had led to deformation in the sewer, and in 1934 it had been decided that strengthening was necessary. That was effected by cutting out considerable quantities of the old brickwork and erecting cast-iron segments, with 3½-inch flanges and internal dimensions as above, inside the old sewer. Voids outside the rings were packed with broken bricks and grouted. The cast iron was lined with concrete flush with the flanges. About 1688 linear feet of sewer were treated in that manner, the flow of sewage, which was about 18 inches deep in dry weather, being maintained throughout the progress of the works. Materials were taken into the sewer through seven small shafts, but lack of storage-space in the street made the works dependent on the daily deliveries of stores. That, and interruption by heavy rains, retarded progress. The cost of the work was about £9·16 per linear foot of sewer.

The use of cast-steel segments at Connaught road was, as pointed out by the Author, unusual. It would be of interest to know why that costly material had been selected, as it would appear to have little advantage

*** This and the succeeding contributions were submitted in writing.—Sec. Inst. C.E.

over cast iron except in its greater strength and elasticity. It was realize that the stability of the Connaught Road tunnels, with their direct load o water and thin cover, was of vital importance to the docks and the low lying neighbourhood, but experience had shown cast-iron tunnels of large arch-spans to be satisfactory, and cast iron had the advantage of being les susceptible to corrosion than steel. The dimensions of the metal o a 16-foot diameter cast-iron tunnel for tube railways would be approxi mately the same as for the 15-foot 8-inch span tunnels described in th Paper.

It would be instructive if the cost of supplying the steel segments coul be given in comparison with the cast iron used in the adjacent pipe subway. The special cast-iron segments used in lining the Victoria Stree sewer cost about £9·13 per ton delivered at site.

The Port Authority was to be congratulated on the great improvemen of the Royal Docks carried out under the Author's supervision, but i was felt that the awkward recess retained at the north-western corner o the Victoria dock might hamper the efficient use of quay-space in tha area.

Mr. John Anderson, referring to the different types of quay-construc tion covered by the Paper, observed that those would form an interestin subject of comparative discussion, divided into false quays, solid gravit quays, open quays with ties or bracings, and open quays withou bracings.

As shown by the Paper, each of those types had merits peculiar to th particular requirements of site and purpose. The first had been used fo providing increased depth of water in front of an existing qua whose foundations were too shallow for deepening alongside. Th monolith quay had been adopted on a site where virgin ground of a poo nature had justified the sinking of monoliths in the dry, as compared wit trench-excavation for a mass-concrete construction. The third type o quay made use of existing and new foundations to provide anchorage b tying back, against the horizontal thrust in the usual way. The fourt type, closely akin to the third, differed in that it was self-supporting withou any bracing members.

Although the first three types were generally well known, he suggeste that the fourth type was unusual, if not entirely novel, and he thought tha a few points in connexion with the north quay of the Victoria dock (*Fig. 1* p. 307) might be of interest.

The conditions under which the unbraced type of quay was adopte for that site were themselves somewhat unusual, for the followin reasons :—

(1) the construction had to be carried out in open waterway with th minimum of interference to the ordinary dock traffic ;

(2) the dock-water at a fixed impounded level limited the constructio free-board to about 5 feet 6 inches from impounded-water lev to cope-level ;

(3) the quay was required to retain pumped reclamation-material subject to consolidation by subsequent pile-driving, without any exterior anchorage.

A block-wall deposited by travelling titan crane and divers might have tisfactorily met all those difficulties, but would have been costly and ower to construct. Other alternatives might have been possible, involv-g preparatory works such as tipping banks for operations to be carried it from dry land, but those would have meant earlier sacrifice of existing rthing facilities. In the actual job as carried out, the temporary work ldom preceded the permanent construction by more than about 3 weeks bout 200 feet in length of gantry). From those general considerations it as evident that the type chosen met the needs of the situation admirably, aving the minimum of underwater work, bracings below impounded-ater level eliminated, and the minimum of robust vertical members tended for easy handling by small cranes.

Piled cylinders, having already proved simple and satisfactory, had een incorporated, with a system of rigid-frame construction designed to e carried out entirely above impounded-water level within the limits of ructural headroom available, and to provide suitable access, under the hole length of the quay, to mains, etc. There were probably few cases monolithic portal construction which were called upon to meet per-anent side-sway thrusts of such magnitude as were imposed upon that iay, in addition to the quite heavy intermittent vertical loadings from anes and railway rolling stock.

Compared with the solid type of quay-wall, it might be pointed out that e horizontal pressure transmitted to the rigid frames, spaced at 24-foot ntres, was only a proportion of the total thrust, consisting of the top action from the sheet-piling and the pressure on the rear cylinders. e balance was taken up by the passive resistance in front of the sheeting, iose relative flexibility ensured independent action between cylinders d the maximum relief to the top thrust. The horizontal reaction of the imes was provided at the foot by the passive resistance of the ground in nt of the cylinders, and the piles driven some 12 or 13 feet below the linders. Partial fixity was assumed at the foot of the cylinders in con-lering the frame-strength.

In connexion with the analysis of the statically-indeterminate frame-esses, it would be impractical, if not impossible, to provide the ideal eoretical conditions of section and profile from cylinder to frame-beam justify highly mathematical treatment. The many assumptions for application of theoretical analysis by methods of slope-deflexion, or east work, etc., being seldom realizable in practice, the results were apt to of purely academic interest, and in the present case the more direct and ite sufficiently accurate Hardy Cross methods of moment-distribution s used. That method had the advantage that by successive approxi-tion, numerical values of bending moment could be quickly computed,

based on trial assumptions of relative stiffness of members, with a view finding the best combination for construction-depth, section of frame-bea and simple arrangement of steel, etc., to satisfy the varying load-conditio

Since the loading conditions would only operate together intermittent the vertical load moments increasing or reducing the value of the horizon thrust moments as the case might be, the permanent and intermitt conditions were considered separately, and the worst combinations w readily found by collecting bending moments according to their positive negative sign. Under the maximum conditions of horizontal thrust deflexion of the quay observable was approximately ¼ inch, which could entirely attributed to the elastic deformation of the frame-members, wh would be expected.

In view of the uncertain quality of the ballast concrete deposited in cylinders by hopper skip through water, and the effectiveness of the st bond, the design stresses in the steel and concrete were naturally kept so what lower than for work above water. Actually there was no evidence hearting concrete being defective. The ballast was carefully tested sand-content, and the quantity of cement was adjusted as necessary secure uniformity of the quality of the mortar.

Referring to the method of construction of the deck, involving s pension from heavy steel girders during concreting (the transverse be ribs already having been concreted), it might be pointed out that the r were designed to act with the slab as T-beams. To make the ribs the selves (with approximately only half the depth of the T-beams) sufficien strong to act as rectangular beams for supporting the dead load of v concrete, would have involved approximately 20 per cent. extra steel.

The extra cost of stiffening the quay section against horizontal thr was actually less than the estimated cost of providing satisfactory exter anchorages.

In dock engineering in Great Britain, mass had previously been t criterion of deep-quay design, often with resultant prejudice to cost. the other hand the design of lighter wharves and jetties in reinfor concrete had tended to follow the lines of similar structures built of timb transverse strength being provided by diagonal bracing or raking pil which, if exposed, were prone to slight accidents, causing local fractu with spawling and baring of steel, and necessitating consequent mainte ance-expenditure.

Those limitations had been successfully avoided in the work un discussion by the co-ordination of modern design-technique and constr tion-methods which were undeveloped 25 years ago, and it might perh be agreed that the best features of mass, economy of design, and proba low maintenance-costs had been retained in the type chosen for the no quay of the Victoria dock.

Mr. R. F. Hindmarsh observed that the Author stated that the lo and passage-way leading to the Royal Albert dock had been reduced depth by inverts rising 4 feet 6 inches to the side walls. The main entra

Northumberland dock on the Tyne had similar inverts at both ends of entrance basin, and a few years ago those were cut down so as to make level bottom right up to the side walls, to suit modern square-built ps. The whole of the entrance-floor and walls were built on piles, so t the invert was not acting as an arch and could be safely removed. e method of doing the work was very similar to that described by the thor for deepening the passage-way between the Royal Albert and Royal ctoria docks. A line of bore-holes was put down with hand jumpers close the foot of the side walls, and formed into a trench by divers, in order to able the diving bell to be placed horizontally over the work ; the bell was ng from a lifting keel and was equipped with the necessary air-pumps, ephone, and electric light. The size of the bell was 9 feet by 7 feet, and weighed 12½ tons ; four men could work comfortably in it. Compressed- tools were used, but were not so effective as was anticipated, the aterial being sandstone. A mason used to that material could do as ll, or better, with his hand tools than the man with the pneumatic rock- ill, which was rather surprising. The whole area was finally roughly essed-off by the mason.

The Author was fortunate in being able to narrow the Royal Albert ck, and so to avoid expensive underpinning of the quay-walls. Why re only two piles driven in the columns, and how far did they go into the ound ? What maximum weight did the columns carry, allowing for the linder sunk to a good hard foundation to take part of the load ? Were e piles timber or reinforced concrete ? Mr. Hindmarsh was particularly erested in that work as he had just designed an 800-foot quay for Tyne ck, where, owing to the ground for a considerable depth being little tter than silt, six and eight piles driven down to hard ground were to used to carry the weight, no bearing value for the bottom of the cylinder elf being taken into account. The cylinders were constructed by means steel sections up to ground-level, 10 feet 6 inches in diameter, and carried foot diameter reinforced-concrete columns from ground-level to deck- el. Removable steel sections of cylinder were first carried up to above gh water, and after pile-driving was finished the bottom was sealed, the linder pumped out and filled with concrete, and the column built in the y. The removable steel sections were then unbolted and used over ain. A substantial timber fendering was provided in front. There did t, however, appear to be any fendering at the north quay widening of the yal Albert dock.

Could the Author say whether the forty-three 3-ton cranes of 60- to -foot radius provided in 1916 on the north quay of the Albert dock, to place the old hydraulic cranes, had been found sufficient in outreach to et requirements ? The outreach beyond the face of the quay would be out from 48 to 53 feet.

Mr. Cecil Peel observed that the scheme for the remodelling of the rth side of the Royal Victoria dock was at present in hand, the construc- n of the warehouses having been commenced and the foundations of

the one at the eastern end completed. Those foundations had invol the driving of three hundred and thirty-four 16-inch, and one hundred a twenty-six 14-inch, square reinforced-concrete piles, 50 feet long, throu the reclaimed ground into the virgin ballast beneath. As stated by t Author, the reclamation filling consisted of ballast dredged from the do and partly dumped and partly pumped over the site of the warehouse a total depth of some 30 feet. The dumping was completed in Ap 1937, and the pumping, which took 3 weeks, in May 1938. Some obser tions upon the settlement of that filling might be of interest. Prior piling the settlement averaged about 1 inch. Pile-driving, which occupi 11 weeks, was completed in October, and during that operation furth settlement took place of an interesting nature. There was an avera settlement over the site of about 1 foot, but in the immediate region of t groups of three or four piles (the groups being 24 feet by 25 feet apart) t settlement was greater, reaching a maximum of 3 feet and averaging inches. In between the pile-groups it was about 6 inches. The drivi was done by a 4-ton semi-automatic steam hammer, falling about 2 fe and finally 3 feet in order to obtain the required set, which caused co siderable vibration and shaking of the ground. Indeed, in places fi ballast was fed around the piles to fill up the depressions in the groun and the vibration caused it to shake down into the interstices of the balla and to disappear. Since the completion of the piling no further settleme had been recorded.

The total displacement of the piles in the filling amounted to about 8 cubic yards, or about 1 per cent. of the volume of the filling on the site the warehouse, and the settlement to about 4 per cent., or a total dimin tion in volume of 5 per cent. It was very satisfactory to note, therefo that in spite of that consolidation and vibration of the ground, which w bound to have increased the lateral pressure, no movement whatever of t new cylinder-quay had been observed, and that provided a valuable che upon the stability of that type of construction. Instances had been kno where quay-walls, particularly of the solid type, had been pushed forwa and fractured as a result of the driving of large numbers of piles at the ba of them, especially when the ground was very firm.

The demolition of the southern ends of the solid jetties in the Roy Victoria dock involved careful procedure, owing to the light constructi of their outer walls, and also to ensure that the debris could be satisfactori removed by dredger. Moreover, owing to the close proximity of buildin and traffic, blasting had to be restricted. The operations involved we mentioned by the Author, and the following was a record of the drilli and blasting. The vault-arches were founded upon continuous concre walls about 4 feet thick, carried down from vault-floor level at + 6·00 N. to the ballast at about — 2·50 N.D. to — 8·00 N.D. The brick outer-vau walls and their concrete foundation-walls, founded on ballast at abo — 4·00 N.D. to — 8·00 N.D., were drilled from arch-springing level + 14·00 N.D., and the inner foundation-walls from vault-floor level,

inch and then 2-inch cruciform drills driven by compressed-air tools orking at a pressure of 80 lb. per square inch. The holes were spaced bout 4 feet 8 inches apart in the centre of the walls, which were drilled early to the bottom. The holes were cleared by a jet of compressed air hich ejected water and débris from them.

The charges were made up with 4-ounce cartridges of Polar Ammon elignite, with L.T. No. 6 electric detonators wired on to laths and lowered ito the holes, the tops of which were tamped with soil. The detonators ere separately tested in a cast-iron pipe before use, using a galvanometer nd battery. The charges were wired in series and connected to a Schaffler xploder, which was operated by the insertion of a key which, on rapid otation through 180 degrees, generated an electric current, contact with he firing circuit being made at the end of the stroke when the current was a naximum. The exploder was tested for firing twenty-five detonators in eries. Before firing, the complete circuit was tested by galvanometer. Io trouble through misfiring had been experienced.

The charges were made up approximately as shown in Table I.

TABLE I.

Depth of hole: feet.	inches.	Charge about 4 feet from top: lb.	Charge near middle: lb.	Charge about 2 feet from bottom: lb.
8	6	—	$\frac{1}{4}$	$2\frac{1}{4}$
10	0	$\frac{1}{4}$	1	$2\frac{1}{4}$
12	0	$\frac{1}{4}$	$1\frac{1}{2}$	$2\frac{1}{2}$
14	0	$\frac{1}{4}$	$1\frac{1}{2}$	$2\frac{1}{2}$
18	0	$\frac{1}{2}$	2	$2\frac{1}{2}$
20	0	$\frac{1}{2}$	$2\frac{1}{2}$	$2\frac{1}{2}$
22	0	$\frac{1}{2}$	2 and 2	$2\frac{1}{2}$

The holes were fired singly, but four holes were charged ahead of he one fired to avoid trouble in charging.

The outer jetty-walls, consisting of 14-inch brick arches with 3 feet of oncrete backing, were demolished in the following way. Divers prized oles in the brickwork and placed charges close to the cast-iron T-piles r at the back of the brickwork. The placing and firing varied at the livers' discretion to suit the way in which the arches collapsed, but enerally four 3-lb. charges were used per 7-foot bay of brickwork, the top ne 1 foot below water-level and the others 7 feet apart. The two top harges were connected and fired together, and the bottom two similarly onnected. The inner-vault foundation-walls were blasted first, in the lry, followed by the outer-vault walls and the jetty-walls, in the wet. The jetty-walls were demolished about 20 feet ahead of the outer-vault valls for the safety of the divers, as the two walls were tied together by ie-rods. All large pieces of jetty-wall which collapsed on to the dock-oottom were re-blasted.

The stability of the jetty-walls during demolition was remarkable in pite of their very light construction.

The Paper provided a valuable comparison of the two types of do construction—jetties and a long straight quay—and the latter appear to have greater merits than the former. With the jetty type of dock mo land and water space was required than with the straight quay, for a wi main dock had to be provided to allow ships to be manœuvred into the berths at the jetties, besides the space required for the jetties and inte mediate "docks." Moreover, considerable economy might be realized the straight-quay type, both in regard to main and subsidiary works. I addition to the quay-walls, roads, railways, and mains had all to be pr vided, and with jetties not only had those to be provided along the jetti themselves, but also along the land at the back to connect the jettie With the long straight type of quay that additional service was obviate and one road and one set of railways and mains at the back of the line sheds generally sufficed. The whole lengths of quay, roads, etc., had fu use and there were no comparatively idle sections at the ends of jetties an the heads of "docks." The accounts of the respective works at the Roy Albert and Royal Victoria docks showed that it was a much easier, quicke and cheaper matter to modernize the straight type of dock than the jett type.

Mr. J. H. W. Turner pointed out that, in the section of the Pap dealing with the deepening of Connaught Road passage, it was made cle that the diving bell was of greater value than the divers. From th contractors' viewpoint, however, he would like to emphasize the advanta of the diving bell in facilitating frequent and easy inspection of the wo being done. Whilst he could strongly recommend it as the best means tackling any similar problems, he would, however, point out that it lo its usefulness where the working surface was other than approximatel level. Before starting work with the bell he had been rather appr hensive about the intensity of the noise which would be caused by usin two pneumatic concrete-breakers inside a small metal chamber, but might be of interest to know that no discomfort had been experience The supply of compressed air for the pneumatic tools was at a pressure 100 lb. per square inch, but since the pressure of 12 lb. per square inc in the bell was also drawn from the same pipe-line, an air-filter was intr duced into the pipe-line in order to make the air suitable for breathin In the pipe-subway the demand for compressed air never exceede 1,000 cubic feet of free air per minute. The incidence of compressed-a illness was very small, and it was of interest to note that its occurren agreed with recent research; namely, that no cases developed until th pressure reached 17 lb. per square inch.

It would be noticed that all flanges on the cast-iron lining had machin faces. In recent years that had become common practice in sub-aqueo tunnels. Where the tunnel was hand-driven it was undoubtedly a improvement, provided that strict supervision was maintained. Whe the tunnel was shield-driven, however, he had come to the conclusio from personal observations, that the extra cost of machining the circu

rential flanges was to a substantial extent lost. That was particularly e case where the shield had a moderate diameter. A shield would show persistent tendency to wander from its true line and level. Where the eviation was small, correction might be made by applying unequal ram-ressures at suitable points around the leading circumferential flange, but doing that there was a tendency to distort the shape of the cast-iron ning. Where the deviation was more serious, some form of packing had to e introduced between adjacent circumferential flanges immediately ehind the shield. Moreover, even if ideal conditions could be obtained or the erection of a length of cast-iron lining, it would be found that the olerances allowed to the segment manufacturers precluded any chance of uilding the perfect tunnel. Slight variations in the widths of different egments would cause unavoidable separations of circumferential flanges. ince, however, ideal conditions for the sub-aqueous tunnel-lining erection hould not be anticipated, he was not in favour of incurring further xpense by imposing finer tolerance on the manufacturers.

In the pipe-subway a large proportion of the bolts were of a special ature to carry the hangers for the various cables and service-pipes. It ometimes happened that a contractor for a sub-aqueous tunnel was nstructed to instal similar fixings after the completion of caulking. He ould like to point out that in doing so the watertightness of the tunnel as liable to be seriously damaged, and that remedial measures might be oth tedious and costly.

The Author, in reply, observed that there seemed to be some confusion ver the method of supporting the beams and decking of the north quay f the Royal Victoria dock (*Fig. 19,* p. 307) during construction. The re-cast beams (Figs. 10, Plate 1) were used as forms for the coping beams nd smaller ones for the centre longitudinal beams. Timber forms were sed for casting in situ the transverse beams up to the underside of the eck-slabs; there was no difficulty in erecting or withdrawing those upports as they were well above dock-water level. Rebates were formed n the centre and transverse beams on which to land 4-inch thick pre-cast labs used as forms for the 12-inch decking. During the placing and ardening of the concrete of the 12-inch decking each transverse beam as suspended from a steel truss 5 feet 6 inches deep by means of four olts, two to each span of the beam, hooked on to similarly placed staple-ars embedded in the rib of the beam. The strain was taken by turning he bolt-nuts until the centre pair gave a part turn with an 18-inch spanner-everage. By observation no uplift took place in the ribs, and a deflexion f about $\frac{1}{8}$ inch was recorded after depositing the wet concrete. The offits of all transverse beams were inspected after stripping, and no fractures r other defects were observed. After the deck-slabs were placed in position ccess to the underside was obtained by the manholes mentioned on p. 308.

No wooden fendering was provided on any of the quays described, but he face of the coping-beams was formed of 12 inches of $1\frac{1}{2}$-inch broken ranite concrete of 1 : 2 : 4 quality, to resist abrasion by barges. The

removal of " I " jetty had greatly improved the access of vessels to " Z berth.

The cost of construction of the typical three-cylinder quay shown i *Fig. 19* (p. 307) including crane and rail tracks, but excluding demolitio of jetties and forming ballast bank, was about £55 per foot run.

The cylinders in the Albert Dock quay were of the standard type show in Fig. 20, Plate 1; they were sunk 2 feet below dredging level and con tained two concrete piles, as the loading was not so great as on the Victori Dock north quay, where three piles at 50 tons were required in additio to the base of the cylinder at 4 tons per square foot on the virgin ballast The deepening of the Albert dock was carried out across the dock to 50 fee off the south quay; the same limit applied to the quay north of th passage.

The outreach of the electric cranes on the north quay, referred to b Mr. Hindmarsh, was up to 53 feet 9 inches, as the front crane-rail wa 4 feet 6 inches back from the quay-face to clear the stem of barges. Tha outreach was sufficient to meet requirements except where vessels of larg beam were " dummied " off about 23 feet to enable barges to be loade between vessel and quay. For that purpose an 18-foot crane-track wa at present being laid down on a portion of the north quay of the Kin George V dock to take 3-ton electric cranes of 80 feet radius.

Referring to the strengthening of the twin railway tunnels, the cost o the cast-steel segments ready for erection was £25 per ton, as compare with £9 per ton for the cast-iron segments of the subway and shafts, bu the greater strength and elasticity of the steel justified its use in th exceptional situation described. The segments were machined by twi millers of American construction.

The pre-cast reinforced-concrete foundations described by Mr. Dean for the roller-paths of the Victoria Dock entrance were reported to hav given satisfactory results until the entrance was closed in 1928 fo reconstruction of the sills.

With reference to the type of construction adopted for quay-walls a the Mudfield, having regard to the loads to be carried and the nature of th materials overlying the ballast, the flexible type of construction wa disregarded. The cost of construction of the main quay-wall, including th excavation within the authorized section shown in *Fig. 12* (p. 302), wa about £65 per foot run.

*** The Correspondence on the foregoing Paper will be published i the Institution Journal for October 1939.—SEC. INST. C.E.

ORDINARY MEETING.

13 December, 1938.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

The Council reported that they had recently transferred to the class of

Members.

Charles James Muir Hunter, B.E. (*New Zealand*).
Peter Murray, B.Sc. (*Edin.*).
Michael O'Brien, C.I.E., B.E. (*Royal*).
Harold Vincent Overfield, B.Sc. (Eng.) (*Lond.*).
Robert Arthur Robertson, B.Sc. (*Glas.*).
William Woodward, B.Sc. (*Birmingham*).

And had admitted as

Students.

John King Andean, B.Sc. (*Edin.*).
Gordon Clifford Keith Baily.
Douglas Francis Belcham.
Robert Harold Charman.
Mark Edward Constant, B.Sc. (Eng.) (*Lond.*).
Kenneth Ernest McIan Daniels.
Dennis Eckersley.
Konrad Joachim Friedlaender.
Jack Rushby Giles.
Peter Noel Grist, B.A. (*Cantab.*).
Frederick Denis Cameron Henry.
John Herbert Hughes, B.Eng. (*Liverpool*).
Reginald John Rees Jenkins, B.A. (*Cantab.*).
Jack Edward Jones.
John Patrick Buchanan Keith, B.A. (*Cantab.*).
Kenneth Henderson Lambert.
Fred Harrison Molyneux.
John Grieve Munro, B.Sc. (*Edin.*).
John Edward Myatt.
Robert Conway O'Flaherty.
John Sidney Platt, B.Sc. (*Manchester*).
John Alfred Price, B.A. (*Cantab.*).
William Rowell.
John Ernest Francis Sandford.
Bhagwat Pershad Sharma.
John Raymond Simpson.
Peter Charles Kerrison Sladden.
Leslie Robert Heron Smith, B.Sc. (Eng.) (*Lond.*)
Geoffrey Spencer.
Arthur Wynford Saxton Stuckey.
Arthur Ernest William Thornton.
Henry George Trimble.
Alan William Keith Tucker.
Anthony Boyse Kelso Tyndall, B.A., B.A.I. (*Dubl.*).
Ronald Leslie Walshe, B.Sc. (*Belfast*).
Herbert Charles Vaughan Woollard, B.Sc. (Eng.) (*Lond.*).

The Scrutineers reported that the following had been duly elected as

Members.

Cecil Rhodes Armitage.
Sir Stanley Vernon Goodall, K.C.B.
Ernest Holloway, O.B.E.
Henry George Howard.
Arthur Mumford Hughes.
Robert Watson Hunter.
Louis Francis Loder, M.C.E. (*Melb.*).
Professor Karl von Terzaghi.
Richard Emrys Thomas.

Associate Members.

Trevor Alwyn Adams.
John Crafts Aldous.
Frederic Clifton Atton, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Ronald Percy Axford, B.Sc. (*Bristol*).
Benjamin Franklin Peter Babcock, Stud. Inst. C.E.
Philip Edward Barrington, M.A. (*Cantab.*).
Maurice Barton, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
William Fred Aspinall Barton, B.Eng. (*Liverpool*).
Charles Hugh Bell.
Max Bentham, M.Sc. (*Manchester*), Stud. Inst. C.E.
Birendra Nath Bhaduri, B.Sc. (Eng.) (*Lond.*), B.Sc. (*Calcutta*).
Alec Bignell, Stud. Inst. C.E.
William Henry Birks, Stud. Inst. C.E.
John Brass, B.Sc. (*Birmingham*).
Thomas Francis Swainston Brass, M.A. (*Cantab.*).
Gerald William Sutton Brown, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Ralph Robert Cambridge, B.Sc. (Eng.) (*Lond.*).
Harry Valentine Carpenter.
Richard Louis Castor, B.Sc. (Eng.) (*Lond.*) Stud. Inst. C.E.
Ganapati Iyer Chellam, B.E. (*Mysore*).
William Edward Chiappe.
Francis Irwin Childs, Stud. Inst. C.E.
George Charles Clark.
William Alan Clarkson.
Cyril Frank Colebrook, Ph.D. (*Lond.*), B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Arthur Richard Collins, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
George Edward Cooke, B.A. (*Cantab.*).
Harry Cooke, B.Sc. (*Manchester*).
Herbert Martin Coombs, B.Sc. (*Bristol*).
Guthrie Stewart Cooper, B.Sc. (*Glas.*), Stud. Inst. C.E.
John Percy Cooper, B.Sc.Tech. (*Manchester*).
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Walter Selwyn Scott, B.Eng. (*Liverpool*).
Frank Thomas Seear, Stud. Inst. C.E.
Parvin Singh Sehmi, Stud. Inst. C.E.
Douglas Whiteside Setterington, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
John Shaw, Stud. Inst. C.E.
Clifford Percy Shelbourne.
Francis Neil Shimmin, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.).
Hubert Ralph Smart.

Douglas William Mead Smith, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Frank John Richard Smith, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
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Jagannath Prasad Srivastava, B.Sc. (*Edin.*).
Charles Harry Stanger.
Geoffrey Dallas Stephen, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.
Kuppuswami Subrahmanyan, B.E. (*Madras*).
William Swan, B.Sc. (*Edin.*), Stud. Inst. C.E.
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James William Wood, Stud. Inst. C.E.
Leslie John Cardew Wood, B.Sc. (Eng.) (*Lond.*).
Horace Edgar Wortley, Stud. Inst. C.E.
George William Alec Wright, B.Sc. (*Cape Town*), Stud. Inst. C.E.
Frederic Henry Merres Young, B.A. (*Oxon.*).
Robert Douglas Young.

Associate.

Charlie Roland Woods, M.B.E.

The following Paper was presented for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Authors.

Paper No. 5198.

"Strata Control in Coal Mines." †

By Harold Taylor Foster, B.Eng., and Michael Anthony Hogan, D.Sc. (Eng.), M. Inst. C.E.

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Introduction.

This Paper deals with some aspects of a research, still in progress, on the problem of supporting the roof and sides of underground excavations. Although final conclusions have not yet been reached it is felt that a discussion of the results so far obtained may be of interest, in view of the somewhat similar problems being studied by the Institution Research Committee (Committee on Earth-Pressures).

The efficient maintenance of the working faces and the roadways leading to them is essential to safe and economical mining. The achievement of a sufficient degree of control over the strata movements to render this possible is of fundamental importance. The term "control" is used because the potential pressures in mines are frequently so great that there can be no question of resisting them by artificial support, and mining is only possible if the strata movements can be controlled in such a way as to free the working spaces from excessive pressure.

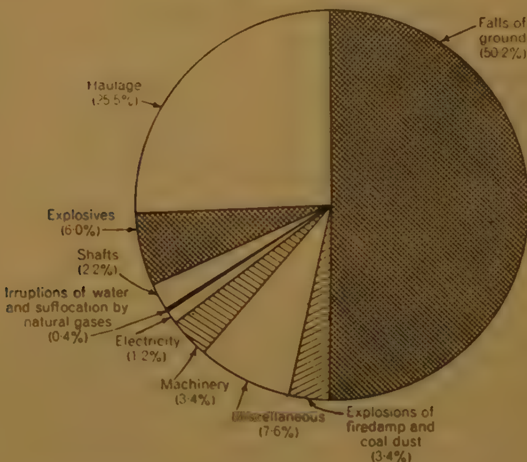
The investigations here described form part of a programme initiated and carried out under the direction of the Safety in Mines Research Board with a view to reducing the number of accidents due to falls of ground. This work commenced in 1923, when a committee was appointed with Sir Richard Redmayne, K.C.B., Past-President Inst. C.E., as Chairman, ". . . to prepare a scheme for investigating possible methods of reducing the number of accidents from falls of ground in coal mines."

[1] Correspondence on this Paper can be accepted until the 15th April, 1939.—Sec. Inst. C.E.

The relative importance of the different causes of accident to underground workers in coal mines is shown in *Fig. 1*, which gives the averag of the three years 1935–6–7. The average number of persons killed o seriously injured each year was 3,701, and 1,856 of these casualties (o 50·2 per cent.) were due to falls of ground. If the comparison is restricted to fatal accidents the proportion due to falls of ground is even higher, amounting to 55·5 per cent.

Besides the accidents directly caused by falls, strata movements also contribute to other types of accidents, for instance on haulage-systems

Fig. 1.



where the restricted working space due to " squeeze " on the roadway may be the proximate cause of the accident, or explosions of firedamp which has accumulated owing to deficient ventilation caused by the restriction of airways.

The theme underlying the scheme of research has been to discover, i possible, general principles governing the movement of strata so as t provide a rational basis for safer methods of mining.

The Factors influencing Strata Movement.

The factors which affect the movements of strata surrounding a mine working may conveniently be divided into two groups : (*a*) natural factors which depend on features existing prior to the mining operations, an (*b*) induced factors, which are consequent on the extraction of the minera

The natural factors include the physical properties of the strata affecte by the workings, the inclination, thickness, and depth of the seam belo the surface, the proximity of faults or folds, the possible existence

ectonic forces in the earth's crust, and the configuration of the surface. Variations in natural factors obviously lead to very diverse working conditions, and any attempt to determine the effect of changes in one factor requires very careful selection of the sites of the experiments. For this reason it is only occasionally possible to follow up the effect of variations in one factor, such as the depth of working, at one mine, but information of this kind may be obtained by correlating the results of observations made at a number of different mines.

The induced factors include the method of winning the coal, the length and direction of the working face, the rate of advance of the face, and the nature of the supports provided. Unlike the natural factors, the induced factors are capable of variation at any given mine.

In Britain two methods of winning coal are adopted. These are known as "longwall" and "bord and pillar," the present Paper being limited to a consideration of the forces and movements met with in "longwall" working.

Types of Investigation.

The experimental investigations fall naturally into four groups: (*a*) observations of the strata movements associated with mine workings, (*b*) observations of the loads carried by supports, (*c*) studies of the physical properties of supports, and (*d*) studies of the physical properties of coal-measure strata.

(*a*) The investigations in the first group serve to define the problem, and to enable the effect of variations in the different factors to be studied. The quantities measured have been the height reduction or convergence, between roof and floor, at the face and on the roadways and the reduction in width of the roadways. The nature and spacing of the roof-breaks and other features are also noted. In some instances the movement of beds lying at a considerable height above the seam have been observed with the aid of a special recording apparatus operated in a borehole. The absolute movements of certain points have sometimes been determined by levelling, but as a rule, use has been made of the measurements of the relative movement of roof and floor.

(*b*) The forces in operation at a coal face are so complex that it is not possible to determine them from the observations of strata movement coupled with a knowledge of the strength (and load–compression relations) of the supports. Accurate information with respect to the loads coming on the supports is required in order that methods of working may be developed in which dangerous fracturing of the roof is prevented.

(*c*) The strengths of the temporary supports used at the coal face and in roadways have been investigated by means of laboratory tests, in which the loading was carefully arranged so as to reproduce the practical conditions met with underground. Steel and wood props, lids and bars, cogs and chocks for face support, and timber settings, and steel and con-

crete arches for roadway support, have been tested in this way. Withou going into detail it may be said that these tests have enabled the relativ efficiencies of different types of support to be determined and have le to a greatly increased use of steel. The outstanding result of these test has been the discovery that the strength of the supports commonly use as set at the face, is only sufficient to carry a weight equivalent to fron 10 to 50 feet of strata, and that the strength of the roadway linings i but little more. In view of their very limited strength, provision is fre quently made for allowing supports to yield at a load less than the breakin load by tapering the wooden props, by using thick wooden lids, or b stilts on steel arches.

In "longwall" working, the term "permanent support" is com monly used to describe the packs which are built in the space from whic the coal has been extracted because these packs are left in, whereas th temporary supports to the roof at the working face are removed wheneve possible. The value of the packs is largely dependent on the nature of th material available for making them; thus where the roof consists c sandstone or hard shale which fractures into approximately rectangula pieces, good packs can readily be built. A great deal of the strength c the pack in the initial stages of loading lies in the walls, and a stone whic lends itself easily to dry building is best for the purpose. Unfortunately in many areas the roof strata are soft and friable, so that sufficient piece large enough for building are not available. Various expedients fo improving the resistance of this class of material, such as the use of bag or wire netting have been studied in the laboratory. The laborator experiments have shown that in the initial stages of loading the supportin power of packs is of the same order as the normal face supports, but th resistance of the packing material rises rapidly with further compressio provided it is retained in position by some constraint, such as tha provided by the pack walls, by burying the packs, or by fallen roof in th waste.

(*d*) A knowledge of the physical properties of coal-measure strata i of value in relation to the type of fracture, the amount of deformatio before fracture, and the reaction between the supports, the roof an the floor following the extraction of a seam. It is also essential if observa tions carried out at different mines are to be correlated, and to enable th conclusions drawn from these observations to be successfully applie Laboratory experiments performed on small specimens of sandston and shale have afforded useful information regarding the strength i compression and bending, and the time-yield of these materials. Othe experiments have shown the way in which the ratio of the lateral t vertical strain (which is known as Poisson's ratio for an elastic materia varies with the magnitude of the vertical load. Experiments on mode roadways, using suitably selected materials, have enabled the type c fracture developed under different assumed conditions to be studied.

MEASURING THE LOADS ON PROPS.

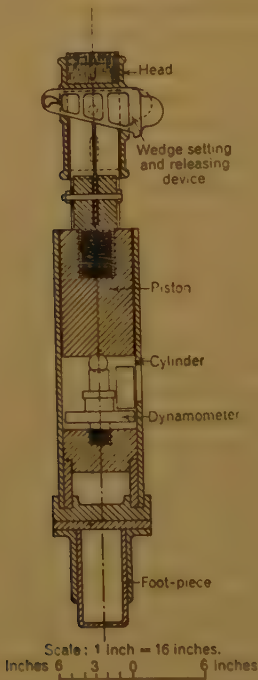
The idea of measuring the loads on props at the coal face with a ynamometer is due to Professor S. M. Dixon, O.B.E., M. Inst. C.E., ho, in 1926, constructed a special type of dynamometer-prop embodying mercury dynamometer made by Dr. G. Wazau. In this type of ynamometer the strain resulting from the action of the load caused anges in the volume of mercury contained within the instrument, these anges being measured by a micrometer device. The actual reading was ade by rotating the micrometer-screw until the mercury rose to an dex mark in a capillary tube. Most of the instruments used in these vestigations were of 100 tons capacity and could be read to 0·1 ton. he total compression of the instrument at full load was of the order of 01 inch, which was negligible in comparison with the compression of her items making up the completed dynamometer-prop, and with the mpression of the neighbouring ordinary props.

In order to protect the dynamometer from injury and to ensure axial ading, it was enclosed within a steel cylinder which served to guide the pper portion of the prop. The internal diameter of the cylinder was bout 8 inches and the piston on top of the instrument was 12 inches ng. This measuring unit was combined with various types of head and ot depending on the kind of props used on the face. The prop illustrated *Fig. 2* (p. 340) was used on a face with "S.F." steel props of 4-inch y 4-inch joist section and was fitted with a standard "S.F." head and a -inch by 4-inch joist foot-piece, the end of the joist being finished by lding over and welding the flanges. Thus the contact areas at either d of the dynamometer-prop were the same as those of the ordinary rops, and consequently the load measured on the dynamometer should e typical of the loads on the props at that face.

The type of information obtained with dynamometer-props may be lustrated by reference to a series of measurements in a seam 4 feet 5 inches ick lying nearly horizontal at a depth of about 250 yards. The seam as overlain by a layer of inferior coal from 10 to 18 inches thick which was ft up to form the roof, and above this was a shale bed with many joints. he floor was a moderately hard clay. At the time the observations were ade, the coal was being won by retreating longwall on a double unit face 8 yards long, the line of face being at an angle of 45 degrees to the cleat. he measurements were made near the middle of one of the 44-yard faces. he coal was undercut daily to a depth of 6 feet. The roof was supported y "S.F." steel props of 4-inch by 4-inch section set in rows from 2 et 6 inches to 3 feet 6 inches apart, the distance between the props a row being 2 feet 6 inches. Sawn timber lids were set above the rops parallel to the face. The cycle of operations commencing with the orning shift was: (1) stripping or filling the undercut coal; (2) coneyor-moving, and withdrawing the two back rows of props; and (3) nder-cutting.

In one test, which was typical of normal working conditions, prop
was set 6 feet from the face at 9 a.m. (during the stripping shift).
initial load of 0·40 tons was developed by driving home the steel wed
when setting the prop, and the load increased gradually as the coal w
shot down and filled away. Prop B was set immediately in front
prop A (2 feet 6 inches from it and 6 feet from the face) when the co

Fig. 2.

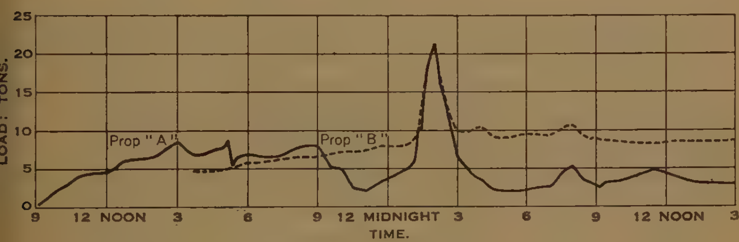


veyor was being moved forward at 3.35 p.m. preparatory to cutting
face. The initial load on prop B was 4·8 tons. The loads on the pr
were observed at frequent intervals (2 minutes when changing rapid
and are plotted in *Fig. 3*. When the back supports were withdrawn
load on prop A began to rise, but this rise was checked by penetrat
of the foot of the prop into the floor. In consequence of this fl
penetration the load on prop A decreased and only began to rise w
the effects of cutting began to be felt. The machine started cutting
the far end of the other panel at 10 p.m., reaching the centre gate
at 11.30 p.m. Cutting recommenced at 12.50 a.m., the machine pas
the dynamometer-props at 1.21 a.m., and reached the end of the face
2.15 a.m. The loads on the props rose gradually until the machine pass

en the rate of increase was greatly accelerated, and the increase conued until both props reached maxima of 21·6 and 21·2 tons respectively, minutes after the passage of the machine. The machine had by then ched a point about 20 yards away. The loads fell off again, nearly as idly as they had risen, due to the penetration of the feet of the props o the floor. The load on prop B remained at 8 to 10 tons whilst that prop A fell to about 4 tons. The adjoining " S.F." props also peneted the floor in a similar manner and the lids were only slightly crushed.

The results of another experiment on another face in the same district the colliery, showing the effect of a " weight " on the face, are plotted *Fig. 4* (p. 342). This face differed from that on which the previous experiint was made in that it was almost parallel to the cleat. The system

Fig. 3.



working and distances between the props were unchanged. The roofditions were very much worse than on the first face and lids were set ove and below the props to increase the resistance. In this test, prop vas set behind the conveyor during the stripping shift, the face was cut that night (the normal cycle being disturbed in consequence of the icult conditions), and, next day, prop D was set 2 feet 6 inches in nt of it and 3 feet 3 inches from the coal. The face was then cut and loads on the props, as the machine passed, reached 19·2 and 18·5 tons pectively. The loads continued to increase reaching maxima of 47·2 l 53·0 tons during the stripping shift. The foot-lids then began to break allowing the props to penetrate the floor, and the loads decreased. height from roof to floor alongside the props had been reduced from eet 6 inches to 3 feet 8 inches mainly by floor penetration. This test ws how the resistance developed by the props was increased by increas- the area in contact with the soft floor. The resistance was in fact ed so much that many of the 4-inch by 4-inch props on the face were t and heavier props of 6-inch by 5-inch section had to be introduced. th the system of support adopted in these experiments, each prop s controlling a roof- and floor-area of approximately 3 feet by 2 feet nches, or 1,080 square inches. The supporting value of the 4-inch by nch props in the first test was limited by the penetration-resistance

of the floor to about 8 tons, equivalent to a roof load of 20 lb. per squa inch, or in other words the props were only capable of supporting a thic ness of 20 feet of strata. The maximum load carried with foot-lids the second test was about 50 tons, equivalent to 125 feet of strata, an in this instance the limiting factor was the strength of the foot-lids.

The rapid rise in the loads on the props on the passage of the coal cutt is associated with a rapid lowering of the roof which can be observed c convergence records taken near the face. The under-cutting of the co

Fig. 4.

has the effect of removing the support from beneath a strip of roof havi a width equal to that of the cut (6 feet in this instance) all along the fa In the particular experiments described, the front prop was 6 feet from t face before cutting, and, apart from necessary temporary props which we removed during the passage of the machine, the front dynamometer-pr was the nearest prop to the face. After the passage of the machine t distance from the prop to the solid coal was 12 feet, and, until the te porary props were reset, a strip of roof 12 feet wide was left witho support. Other experiments gave similar results and confirmed t importance of providing supports at the earliest possible moment to res the impulsive load set up by undercutting the coal.

Measuring the Loads on Packs.

The problem of measuring the loads on packs bears some resemblance ɔ that of measuring the bearing pressures under foundations, but whereas ıe maximum pressure on foundations is of the order of from 4 to 6 tons er square foot or from 60 to 90 lb. per square inch, pressures of over ,000 lb. per square inch have been measured on packs. Consequently, pparatus of very much higher capacity was required for the pack-load ıeasurements. In developing this apparatus numerous preliminary ex- eriments were made in the laboratory in which various forms of dynamo- ıeter were used to measure the loads at a point in an experimental pack, r filled cog, subjected to load in a 400-ton testing machine.

The first underground experiments were carried out with the object f studying the initial resistance offered by the packs immediately on ɔmpletion in the early stages of compression, when they could be regarded s taking the place of face supports. These experiments were intended to erify the laboratory experiments on packs which had shown that the ıitial resistance was of an unexpectedly low order. After a few trials had een made with instruments which were of limited capacity and had to e taken out within a short distance of the face, it was decided to use ıstruments of larger capacity which could measure the pressure in the acks at a considerable distance from the face.

An earlier attempt to measure the loads on packs was made by Dr. [. Hoffman[1] in 1933, using a steel channel supported on two mercury- ell dynamometers. The dynamometers stood on the floor and the ıeasuring apparatus projected into the pack. In view of the fact that ıe measuring apparatus was much more rigid than the packing material, seemed likely that this arrangement would give pressures considerably igher than the true value. This anticipation was verified in a laboratory ɛst on a pack of soft shale, when the unit load recorded on the dynamo- ıeter by Hoffman's method was from $1\frac{1}{2}$ to 3 times the average load pplied. As a result of this experiment the succeeding dynamometers ere sunk so that the top of the instrument was flush with the floor. ; was decided also to measure the pressure on circular areas sufficiently ırge to avoid error by irregular loading due to the action of large packing ıaterial.

In the laboratory, good results were obtained when a 9-inch diameter eel disk was set flush with the surface of the roof or floor in the testing ıachine and supported by a Wazau mercury-cell dynamometer. This ype of instrument was, however, not very suitable for use beneath a pack nderground, on account of the difficulty of adjusting the micrometer rew and viewing the mercury thread at a distance. It was accordingly ecided to develop a simple type of dynamometer which could be read ore easily under the conditions of the pack-load experiments. The

[1] *Thesis submitted to* Technische Hochschule, Aachen, 1933.

type of dynamometer adopted for the first underground tests, which was devised by Mr. W. H. Evans, of the Safety in Mines Research Board staff, is shown diagrammatically in *Fig. 5.* The load causes a compression in the vertical steel cylinder A (about 4 inches external diameter and up to 1 inch thick), and this compression is transmitted to the bent steel strip B through the medium of ball-joints C. The strip is set with an initial curvature and the compression causes a lateral deflexion of the middle

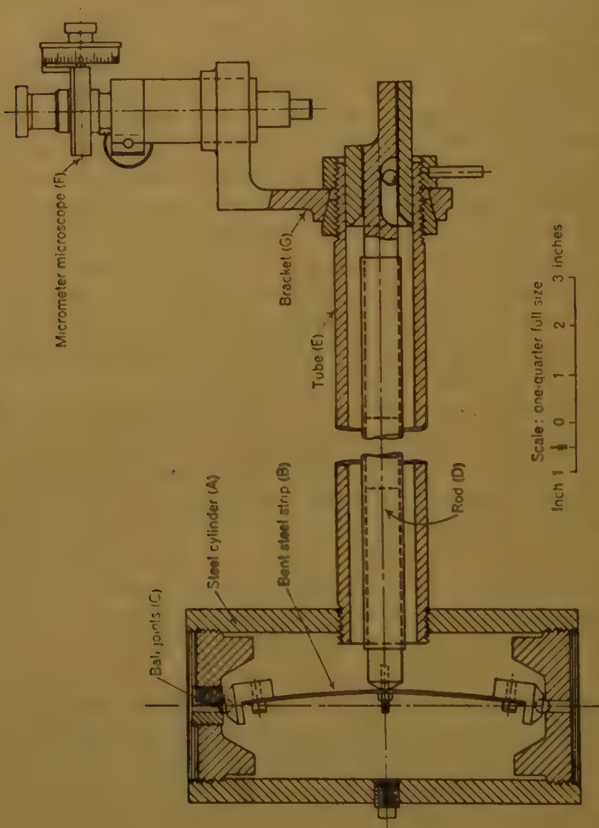


Fig. 5.

of the strip which is about 4 times the strain in the cylinder. Th lateral deflexion is transmitted by the rod D to the measuring point, whe the movement of the rod can be checked with reference to a tube E fixe to the cylinder. The movement of the rod D was measured by mea of a portable micrometer-microscope F, which was mounted on a speci bracket G to secure accurate registration. Three cells were used, the being placed vertically between two large steel disks, 13½ or 18 inches

ameter. The upper disk, which was flush with the floor of the seam, as carried on the cells through ball-joints so that the cells remained naffected by any deflexion of the disk.

In the more recent instruments the cell bodies were made of nickel-romium steel, and with these the readings could be taken to within 3 per cent. of the total load. The maximum length of rod used with these struments was 9 feet, and since the operator required a space of about 3 inches at the roadside for reading, the distance of the instrument from e side of the pack was limited to about 7 feet 6 inches.

Some difficulty was experienced with the reading of the micrometers nderground, and a direct reading form of dynamometer has been developed which the indications are given by a dial gauge. In this instrument the rain in a single cylinder about 10 inches diameter operates two deflecting rips, and is indicated on the gauge by their lateral motion. The distance which this type of dynamometer can be read depends on the illumina-on and the power of the telescope, and with the arrangements used in e underground tests, satisfactory readings can be made at a distance of feet. Adequate steps must be taken to keep a clear line of sight from e roadside. For this purpose a steel tube 4 or 6 inches diameter is used, d this tube can be protected by heavy steel sections. The precautions ay, however, defeat their own end because the disturbance of the floor used by the excavation for the tube and its protecting steelwork may ovoke floor movements that eventually block the line of sight (this ouble has been met with at depths of the order of 600 yards).

In view of the difficulty of maintaining a line of sight an electrical pe of dynamometer has been developed. Cylindrical steel cells are ain used to carry the load, the strain being measured by the change resistance of a pair of coils of fine steel wire which are mounted inside e cell. The cells are filled with oil to protect the wires from sudden anges of temperature and the change in resistance is measured on a idge circuit with a portable galvanometer-set. With this type of instru-ent the dynamometer can be set at any point in the pack provided the ads to the measuring point can be protected from damage.

The Results of Pack-Load Measurements Underground.

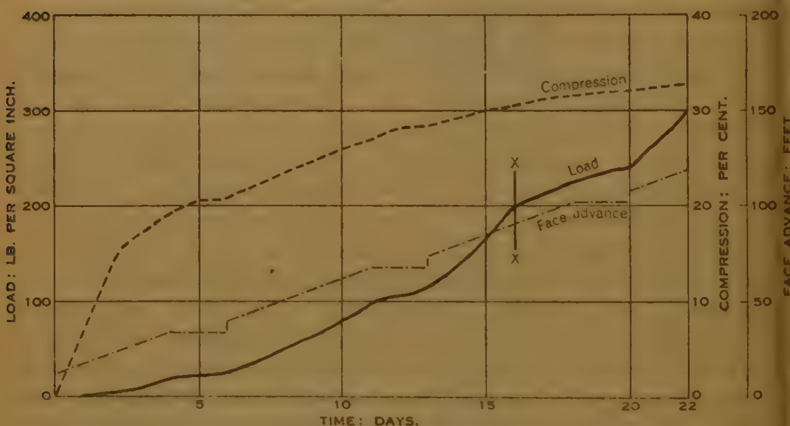
In the underground experiments, the dynamometers were set as the cks were being built close to the coal face, and readings were taken at tervals depending on the rate of advance and the distance from the ce. At the same time subsidence recorders were set to measure the con-rgence of the roof and floor at the roadside, or in a refuge-hole close the point of measurement. In recent experiments convergence measure-ents have also been made at points within the pack by means of special uges which enable the reading to be made at the roadside. Thus the cords of an underground pack-load test include a series of pressure

readings taken after different intervals of time, with corresponding con-vergence measurements, and measurements of the distance from th instrument to the face. In some instances the width of the roadway was also recorded.

It is not necessary in this Paper to discuss a large number of detailed observations, and a few typical records have been selected to show th pressures obtained at different mines under various conditions of deptl and strata movement. The results of the pressure measurements can b presented in several ways: the observed pressures can be plotted agains either the time, the distance from the face, or the pack-compression Examples of these three methods of plotting follow.

The effect of plotting the readings to a time-base is shown in *Fig. 6*

Fig. 6.



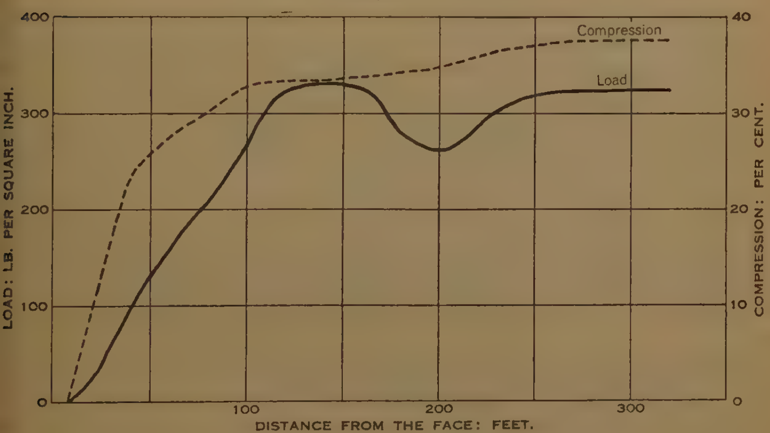
which relates to observations taken with one of the early instruments i a seam 4 feet thick at a depth of 250 feet below the surface. The face wa 113 yards long and there were gate side packs 5 yards wide with 2-yar intermediate packs and 10-yard wastes, the area packed being approxi-mately 42 per cent. of that excavated. The dynamometer was set a the side of a roadway in the middle of the face, and 2 feet 6 inches from th side of the pack when it was being built at a distance of 12 feet from th face. The pressure [1] rose very gradually at first, and after the sixth da when work was resumed following a week-end stoppage the curve showe

[1] The pressures have been measured in lb. per square inch. A pressure of 1 l per square inch is equal to the dead weight of a column of rock 1 foot high if the roc weighs 144 lb. per cubic foot, equivalent to a density of 2·3. As this value is ve close to the average density of the overlying rocks, it may be taken that the stat pressure due to the overlying rocks is 1 lb. per square inch per foot of depth, so th the pressure in lb. per square inch is equivalent to the height in feet of a column strata which would cause that pressure at its base.

steepening. During the second week-end, on the eleventh and twelfth ıys the pressure curve eased off, and resumed its former direction when ork began again. The pressure curve showed a check on the seventeenth ıy (marked XX) due to the partial collapse of the pack-wall in front of .e dynamometer, but after this had been rebuilt the resistance curve sumed its former upward trend. The experiment came to an end on .e twenty-second day because the pressure then reached the maximum pacity of the dynamometer. It will be noted that the pack–compression ırve rose most steeply during the first few days before the pack had gun to develop any appreciable resistance, and at the first week-end .e compression was 22 per cent. of the original height whilst the load ı the pack was only about 20 lb. per square inch.

A somewhat similar experiment, plotted to a base of distance from the ce, is shown in *Fig. 7*. The face was 180 yards long in a seam 37 inches

Fig. 7.



ick lying at a depth of 200 feet below the surface. The dynamometer as set 3 feet from the side of a 5-yard pack on the centre roadway whilst .e pack was being built at a distance of 8 feet from the face. The face ıd a solid coal rib on one side and old goaf on the other. Besides the yard roadside packs there were 3-yard intermediate packs with 6-yard astes, the area packed being 40 per cent. of the whole. In this experiment .e load rose steadily and reached a maximum of 333 lb. per square inch ter 55 days when the face was 144 feet away. It then fell slightly to minimum of 260 lb. per square inch at 197 feet from the face, and rose ain, reaching 322 lb. per square inch at 265 feet from the face. The essure remained at this value until the face was 315 feet away, when it ll off slightly to 302 lb. per square inch at 460 feet from the face. In this se the roadway was supported by steel arches and the pack-wall did not

collapse. It will be noted that the maximum pressure recorded in this test was 1·66 times that due to depth of strata.

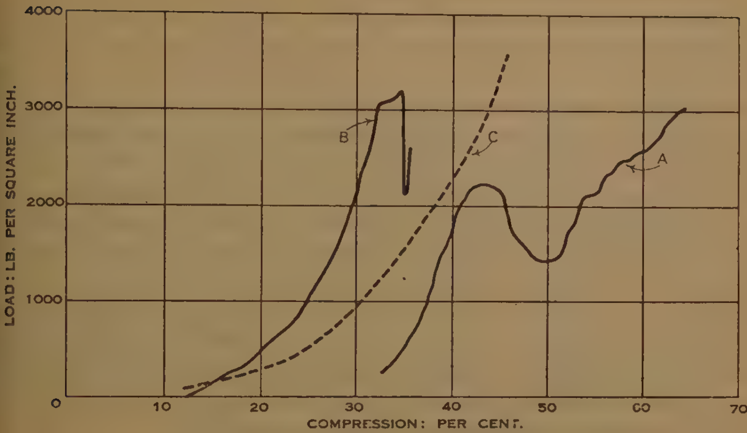
In subsequent experiments on a similar face in the same seam three dynamometers were set across the pack, one in the middle and one 2 feet 6 inches from each side. It was found that the instruments at the roadside and waste sides gave practically identical records which differed little from that shown in *Fig. 7*. When the face had advanced about 100 feet and the pressure was in the neighbourhood of 300 lb. per square inch, the curve for the centre dynamometer took a sharp turn upwards, reaching a maximum of 915 lb. per square inch, 2½ times that of the two other instruments. The maximum loads on the three dynamometers occurred at practically the same distance from the face, showing that in this particular seam the pressure varies considerably across the pack and that the loading on the pack is substantially symmetrical. The fact that the load varies so rapidly with the distance of the instrument from the side of the pack indicates that any record taken with a single instrument is to be regarded as of qualitative value only.

Similar variations in pressure have been observed across a pack in a seam ranging in thickness from 4 to 5 feet at a depth of 1,350 feet, and in this case measurements of pack-compression made at the centre of the pack showed that the movement there was considerably less than close to the side. Thus the higher pressure at the centre of the pack appears to be associated with the better supporting power of the material due to the lateral constraint imposed by the surrounding material. Close to the sides where lateral movement is possible the pack yields more easily and the resistance is less. At the edge of the pack the resistance will obviously be greater in the earlier stages of compression due to the action of the wall.

Records have been taken at a depth of 1,980 feet in a 9-foot seam, of which only the lower 6 feet were worked. The instrument was set in the middle of a 12-yard roadside pack near the centre of a face 400 yards long with 10-yard intermediate packs separated by dummy roads 8 feet wide, 70 per cent. of the area being carried on packs. In one typical experiment (curve A in *Fig. 8*, where the load is plotted against the pack-compression) the pressure reached an initial maximum of 2,200 lb. per square inch when the face had advanced 169 feet from the instrument, and then decreased to a minimum of 1,450 lb. per square inch 210 feet from the face. Thereafter the pressure rose steadily until it reached the limit of the instrument's capacity and the test was stopped when the pressure was 3,015 lb. per square inch 847 feet from the face. This pressure was 1·52 times the static pressure due to the depth.

The records taken at very different depths have one characteristic in common—a fairly rapid rise of pressure to a maximum value followed by a fall and then a further rise. In the observations taken at a depth of from 200 to 250 feet the initial maximum was the highest pressure recorded,

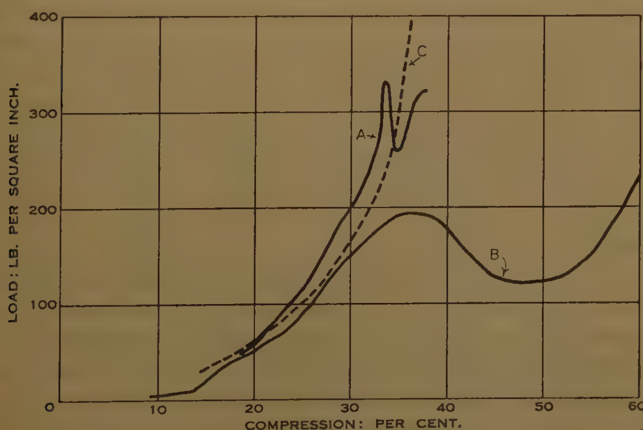
Fig. 8.



ut at a depth of 1,980 feet the initial maximum was considerably below ie pressure subsequently reached.

In order to arrive at an explanation of the cause of this variation in ressure it is necessary to consider the relation between the load and ompression on the pack. When these quantities are plotted for tests arried out at shallow depths a gradual increase in pressure with increase 1 compression is seen. The curve frequently stops at the initial maximum oad, indicating that no further compression takes place. In some instances, owever, as in the case of test already described and plotted in *Fig.* 7, the urve shows a drop and subsequent rise again (curve A in *Fig.* 9). This ype of behaviour has been observed in all the tests carried out at depths f the order of 600 yards.

Fig. 9.



The results of a test at a depth of 1,050 feet are plotted in *Fig. 8* (p. 349 (curve B) for comparison with curve A, at 1,980 feet, previously described The instrument was set beneath the middle of the pack in both instances but in test A the compression was measured in a refuge-hole 10 feet from the instrument and 5 feet from the side of the pack, whereas in test I the compression was measured in the middle of the pack close to th dynamometer. The compression was considerably greater in the refuge hole than in the centre of the pack, in test A, because at the end of th test the compression in the refuge-hole was 64 per cent., but on diggin out the instrument the compression close to it was found to be only 4 per cent. Test B was unfinished and the pack-compression had only reached 36 per cent. when this Paper was written, the compression nea the roadside being then 62 per cent. This confirms that the compression in the middle of the pack was less than close to the sides. At the middl of the pack the drop in pressure after the initial maximum load is accom panied by only a very slight increase in compression.

It is suggested that the drop in pressure following the initial peak i to be attributed to the failure of the pack or of the floor strata with which it is in contact. On the roadside this failure may be masked by the action of the supports, or in certain cases may make itself felt in the form o pushing in of the sides, or rising of the floor. On the waste side it probably means the collapse of the pack-wall. When failure of this kind occurs th pack will yield rapidly, until some constraint comes into action which pre vents further yield. Constraint of this kind may be furnished in practic either by the supports at the roadside, by the curvature of the roof strata by fallen roof material on the goaf side, or by taking a thick ripping an completely burying the packs beneath the roadway. As soon as th lateral spreading of the material is stopped the resistance will begin t rise again. An effect of this nature has been noted in some of the laborator tests on packing materials, and curve B in *Fig. 9* (p. 349) shows the same typ of initial maximum followed by a drop and gradual rise. This curve relate to the test of a bag-cog 3 feet square and 4 feet 8 inches high, which reache an initial maximum load of 192 lb. per square inch with 30·6 per cent compression ; the load then fell to a minimum of 112 lb. per square inch and at this stage the bulging material at one side of the cog came in contac with a vertical wall at one side of the testing machine. The wall afforde the cog support and prevented further collapse on that side, so that th load rose again until the test was stopped at a pressure of 224 lb. pe square inch.

This experiment illustrates the effect of lateral constraint in improvin the resistance of a pack after the initial maximum load has begun to caus collapse. It is realized that the bag-cog used for this test was very much weaker than a pack underground, owing to the greater constraint afforde by the friction between the pack, the roof, and the floor, particularly when the roof has begun to bend over the pack and the pack to push int

ıe floor, but the result is of interest as a possible explanation for the ırious behaviour of the load–compression curve at considerable depths.

It may be remarked that the very slight variations shown by the load–ompression curve for the tests at shallow depths (curve A, *Fig. 9*, p. 349) orrespond to the relatively stable behaviour of the packs and roadway ıring that test. The drop and rise in the pressure curve probably mean ıat there was some slight failure of the pack-walls, which brought them into ontact with the steel arches lining the roadway before much displacement ıd occurred. In the tests at a depth of 1,980 feet the floor was weak and ıe drop from the initial maximum pressure coincided with widespread oor movements, from beneath the pack into the roadway. The pressure ıly began to rise again when these movements had been checked by pair work.

The very different resistances developed by the packs in the tests at edium and shallow depths will be noticed on comparing the curves in *Figs. 8* ıd *9* (p. 349). The difference is explained neither by the nature of the aterial used for packing in the two cases, nor by differences in the method building the packs. Similar differences in resistance have been recorded some laboratory tests carried out by the U.S. Bureau of Mines[1], and it appens that two of these tests gave results closely corresponding to the oserved behaviour of the packs in *Figs. 8* and *9*. The rock used in these sts was that found in association with the anthracite seams of Pennsyl-ania, and appears to have been a fairly hard shale. U.S. Bureau of Mines est No. 6 on a " Circular mine-rock cog, laid up of dry rubble masonry, ntre filled with small broken mine rock, ashes and culm. Rock mostly nall pieces . . ." gave results comparable with those obtained in the deeper am, and are plotted in *Fig. 8* (p. 349) (curve C). This cog was 4 feet in ameter, and 1 foot 6 inches high. U.S. Bureau of Mines Test No. 13 on " loosely laid-up mine-rock cog, voids filled with fines from a previous st and one half sand and one half ashes for centre filling " gave results hich are comparable with the load–compression curves for shallow depths shown in *Fig. 9* (p. 349) (curve C). This cog was 2 feet high, 5 feet uare at the bottom and 3 feet square at the top. The difference between ıe results of these two tests is probably due to the fact that larger stones ere used in the building of specimen No. 13, so that it was not so well ompacted as No. 6. Thus these laboratory experiments confirm the ependence of the quality of a pack on the quality of the material used in ıilding it.

The observations which have been briefly reviewed in this Paper show ıat in modern machine-mining the undercutting of the coal causes a pid increase in the load on the face supports, which is accompanied by a arked acceleration in the subsidence of the roof. This movement is pparent along the face, well in advance of the machine, and can be detected to a distance of from 20 to 35 yards back from the face. The forces

[1] U.S. Bureau of Mines Bulletin No. 303. 1929.

developed during coal cutting in many instances lead to fracturing of the roof, and methods of support are required which will prevent or diminish the extent of these fractures and the excessive roof movements associated with them.

Conclusions.

The load on the supports at the coal face bears no definite relation to the depth of the seam, and it is suggested that the load more probably depends on the thickness of the beds which become detached from the superjacent strata in the region of the working face. The thickness of this detached zone will depend on the nature of the strata above the coal seam and the method of working. It may be assumed that the pressure above the detached zone is carried by a stratum strong enough to span the distance between the solid coal and the point behind the face at which the permanent supports become effective. Even within the detached zone, the beds, although they may be fractured, will not be altogether devoid of supporting power, and the force required to keep them from falling will not be very great. The idea of a detached zone enables the face supports to be designed on a definite system. If the height of the detached zone is known, the total load to be carried by the face supports may be calculated, and thus the load on each individual support arranged on a suitable system of spacing can be obtained. With this knowledge and that of the penetration resistance of the roof and floor, the dimensions and contact-areas of the supports can be ascertained, so as to provide effective strata control without causing local damage or fracturing of the roof or floor. The prevention of local fracturing, due to the use of supports with insufficient contact-areas, is of considerable practical significance because many accidents are caused by unequal loading and the development of excessive local stresses on individual supports.

The same principle of avoiding high local stresses applies in the case of the packs. Under present-day mining conditions it is difficult to provide a uniform resistance to the general subsidence over the excavated area, and excessive stresses may be induced in the strata at points where the supports develop a high resistance. This frequently occurs at roadsides leading to collapse of the pack-wall or to heaving of the floor, as shown in the present tests, and a special method [1] of packing has been devised to overcome the difficulty. In this method, the main packs are situated at a distance of several yards from the roadsides so that the fracturing of the roof or floor caused by the high resistance of the pack will not damage the roadway. Subsidiary narrow packs are built on either side of the roadway to carry the immediate roof strata, but on account of their lower resistance (due to the reduced width) these packs do not cause fracturing of the roof or floor strata.

[1] Annual Report of the Safety in Mines Research Board, 1937, p. 27. H.M. Stationery Office.

Acknowledgements.

This Paper is presented by permission of the Safety in Mines Research Board, and the Authors wish to thank Professor S. M. Dixon, M.A., B.A.I., O.B.E., M. Inst. C.E., and Major H. M. Hudspeth, D.S.O., for their interest and advice in the earlier stages of the work. They are also indebted to Mr. W. H. Evans for the detailed design and construction of the dynamometers used, to Messrs. H. Henshaw and E. H. Vallis who arranged and carried out the underground observations, and to the managements and staffs of the collieries who assisted in the measurements.

The Paper is accompanied by nine sheets of drawings, from which the Figures in the text have been prepared.

Discussion.

Dr. M. A. Hogan, in presenting the Paper, emphasized the fact that it was a preliminary report upon a research which was still in progress, and said that the Authors hoped to learn from the discussion ways in which the measurements might perhaps be improved.

The dynamometer prop shown in *Fig. 2* (p. 340) had been abandoned in subsequent work because of the complication involved in having a special type of prop, and ordinary steel props were being used, measurements being made with a strain-gauge. The mercury dynamometer could be used underground because at the coal face the temperature-changes were small, but if it were used at the surface it would act as a thermometer and the readings would be erratic.

Mr. F. E. Wentworth-Sheilds expressed the regret which he knew was felt by all those present that Sir Richard Redmayne, K.C.B., Past-President Inst. C.E., was unable to be with them that evening, as he had played a very important part in the commencement of the research.

At the beginning of the Paper the Authors expressed the view that a discussion of the results so far obtained might be of interest in view of the somewhat similar problems being studied by the Institution Research Committee. He was quite sure that every member of that Committee would agree with that statement, although it could not be said in any sense that the researches overlapped. The problem of the mining engineer was how to support an enormous slab which was over his head, whilst the problem of the surface engineer was how to distribute his load on that enormous slab. The Committee on Earth-Pressures of the Institution Research Committee had, during the past few years, been studying four main problems. The first was the classification of soils according to their physical and mechanical properties, such as friction and cohesive strength; the second and third problems were the estimation of the movement of soils under vertical and horizontal loads respectively; and the fourth problem was the estimation of forces such as arose when slips were formed or landslides took place. In the first research much useful work had been done in different countries, and a technique had been evolved for ascertaining the mechanical properties of different soils, and therefore of classifying them. In regard to the estimation of settlement under vertical load, some very successful work had been done, and a stage had been reached where it was possible more or less to estimate what vertical movement a given load would produce on a known soil. With regard to the problem of movement under lateral pressure, the work had not gone so

far; some fine work had been done on the lateral pressure of sand, but on the lateral pressure of materials which depended not only upon friction but also upon cohesion for their strength the work was very complicated, and there was a great deal yet to be done. Some very useful practical results had been obtained on the problem of stability under tendency to slip.

The surface engineer could learn a great deal from a Paper such as the Authors had presented. Packs, for example, which seemed at times to consist of dry rubble masonry and sometimes of what might be called sacks of rubbish, were able to carry immense loads. That brought home to surface engineers the problem of retaining walls. Miles of expensive retaining walls had been built of concrete or brickwork, no part of which was stressed to anything like its safe limit; such structures were clearly grossly uneconomic. Of late years an endeavour had been made to remedy that lack of economy, and one result was the L-shaped reinforced-concrete retaining wall, which bore considerable stresses. Another result was that type of quay-wall which was formed by sheet-piling supported by framing. He was hopeful, however, that a type of wall would be produced which would be made of cheap material and which would be stressed so that it would transmit its load without excessive settlement. Increased transport meant building lines of communication with easy gradients and flat curves, and therefore involved large earthworks, where a wall of that type would be very valuable.

Sir Henry Walker remarked that he had been connected with mines for 40 years, and had had the awful toll of accidents from falls constantly before him. He would like briefly to refer to the historical aspect of the work. The committee referred to on p. 335, had, he believed, done nothing, and in 1924 it had been reappointed with a different constitution. His object in mentioning that was to honour the late Sir Thomas Mottram, C.B.E., who was the Chairman of that Committee; it did the initial work for the researches which followed. He (Sir Henry) had been a member of that committee, which worked until 1928, and visited each district. It should be mentioned that the Royal Commission of 1908 appointed a Committee consisting of Sir Richard Redmayne as Chairman, the late Mr. A. N. Lamb, and the late Sir William Walker (Sir Henry's brother), but their Report, which dealt with accidents from falls and from other causes, was more or less a description of practice. After the report of the Mottram Committee in 1928 those members of the Safety in Mines Research Board who were mining engineers thought that the time had come when part of the money spent by that Board should be spent upon practical researches, and their request was agreed to. Since *Fig. 1* (p. 336) showed that more than half the accidents which happened underground were due to falls, he thought that it would be agreed that the request was well justified. In 1928 an Inspector of Mines, Major H. M. Hudspeth, D.S.O., M.Sc., was appointed as Mining Engineer to the Board, and it was due to him to say that he organized the work of research throughout the coalfields in Great

Britain. About a year ago Major Hudspeth was succeeded by Mr. H. T. Foster, one of the Authors of the present Paper.

On p. 352 the Authors stated that if the idea of a detached zone were accepted, the face-supports could be designed on a definite system. Sir Henry did not see any difficulty in accepting that idea, but the ability to design face-supports depended, as was stated in the Paper, amongst other things on the height of the detached zone, and he would like to know how that height was to be ascertained, and, when it was ascertained for any seam, whether it would hold good for that seam throughout a single colliery, or even throughout a single district of a colliery. Would it be correct to say that if the height of the detached zone could be ascertained the problem of roof control might be considered to be solved, always assuming the provision and proper use of adequate materials and good workmanship ?

Professor J. A. S. Ritson observed that the first point which he would like to emphasize was with regard to the packs. A well-built pack should be built with flat stones which should be bonded like brickwork, and the inside bonding should be in intimate contact with the filling, which should consist of as small stones as possible. While it was perfectly true, as the Authors pointed out, that the walls did give initial support as the coal face advanced, he thought that it was also equally true to say that the main support came not from the walls but from the filling. The walls were really retaining walls to prevent the filling spreading during compression. As the coal was removed, especially in the longwall system of mining, all the ground was in a state of movement, and that movement began in advance of the coal face and continued for a considerable distance back from the face ; the movement was largely downward, and it had to be arrested by the pack-filling before equilibrium was restored. The sooner the movement was arrested the better, and therefore the pack walls should be built as close as possible to the working face, since the ground would then have less time to settle and fewer fractures would occur.

The roofs in some mines were so friable that they did not provide good flat stones for pack-wall building ; attempts had been made to overcome the difficulty by using sandbags or wire netting, and in South Africa, in particular, wire netting of quite a heavy wire gauge was used to retain the outer walls in position. On the other hand, a very common reason why flat building stones were not obtained was that no consideration was given to the use of the proper kind of explosive in breaking up the ground to make sufficient room for the men to work. There was no doubt that to-day, with the many types of explosive available, it was possible, with careful use of the explosive, to break the ground sufficiently to enable it to be removed, and yet to provide stones of the proper shape and size for building the pack walls. Far too frequently, however, the explosive used was too powerful or too rapid, or more frequently still, too great a weight of explosive was used for the burden against which it had to work.

As was made clear by the Paper, no temporary supports could carry the weight of the overlying rock, and therefore mining engineers had developed the conception of an arching or corbelling effect, and at the coal face they assumed that one abutment of that arch rested on the unwrought coal and the other abutment rested on the packs which were in process of consolidation behind. It therefore followed that the quicker the pack upon which the abutment rested became static the less would be the height of the corbelling effect and the less the intradorsal pressure to be carried by the temporary supports. The obvious corollary from that was that immediately coal was undercut by the machine it should be supported, and that the pack walls should then be built as quickly as possible.

The Authors gave figures which showed that varying pressures were found within the packs, and they also showed that the maximum pressure as measured within the packs was greater than the calculated pressure due to the overlying rocks. Those packs were not continuous throughout the mine; they were made at right angles to the face with certain distances between them, and therefore there were zones between them which were unsupported and into which the roof ultimately collapsed. The very heavy pressures which were found on the packs some distance back from the face would therefore appear to be due to the formation of more arches, with their axis parallel to the face, and that therefore the increased pressure which was obtained on packs from 100 to 200 feet back from the face would be due to the abutment-pressure of such arches. It also showed that the roof which had fallen between the packs had not consolidated, and therefore was not taking its full share of the weight. Under those conditions the area between the parallel packs was likely to become a reservoir for firedamp. The resistance of the pack walls, according to the Authors, would be greater during the initial stages of the compression, but how long did that actual resistance last? When did the transition of pressure from the wall to the filling take place? On his submission that the filling was the real supporting material, could that transition-time by any means be reduced?

The most difficult ground which was met with in coal mining occurred when the ground began to swell, due either to the effect of water or to moist air. Many main roads in mines were to-day adequately supported by steel arches which were not sufficient to carry the weight to which they might be subjected. That was probably due to the fact that local arching took place over those main roads and that the pressure was taken on the sides and not by the steel arches at all. If the steel arches in many main roads in mines were removed once the ground had become stable, the roads themselves would probably stand perfectly well, provided that the ground was not affected by weathering. On the other hand, there were roads which, however well they were supported, always gave trouble, and there again that difficulty had been overcome in many cases not by the strengthening of the supports put in the road, but by preventing the action of

weathering on the roads. Lining them with thin brickwork or sheet metal had undoubtedly had a very great effect in preventing deterioration of the ground.

The pressures which had been mentioned by the Authors were undoubtedly fairly high, but on the other hand they did not compare with the pressures which were met with in deep gold mines; in the Witwatersrand, for example, mining was proceeding at a vertical depth of just under 9,000 feet from the surface, although the rocks were admittedly more homogeneous and stronger than coal strata. The first stage in mining at that depth was to divide the valuable portion of the deposit into large pillars, roughly 150 yards square, and then those pillars were extracted by longwall methods which approached towards the centre of the pillar and eventually formed a small triangle. That triangle, or "remnant" as it was known in South Africa, was perhaps the most difficult piece of ground to extract which had, as far as he was aware, yet been attempted by mining engineers; the pressure was such that pieces flew off the remnant with more or less explosive violence, and when settlement ultimately took place and a fracture occurred, a considerable amount of damage was done for a distance of perhaps 100 yards around the remnant. That difficulty had been very largely minimized, both by careful observation of the rate of subsidence of the roof, using instruments very similar to those designed by one of the Authors of the Paper (the instruments being read at half-hourly intervals when nearing the critical time), and by the very extensive use of carefully built dry-stone walls or packs. In the mines in question the work was carried out with unprotected lamps, and the shock caused by the breakage of the remnant invariably put those lights out; quite a number of accidents had been caused in the minor panic which invariably followed a rock-burst of that type. Such accidents had been to a very large extent eliminated by the use of electric lamps, which were not affected by the blast which occurred, and a row of which was always hung up in the district where remnants were being removed.

Dr. Herbert Chatley remarked that the work should lead to further knowledge on the subject of arching over cavities, to which reference had already been made. If a small cavity were made in a large loaded mass the load would be carried completely over the cavity and there would be no perceptible roof-pressure, but as the cavity became larger the position changed and reinforcement was necessary in the roof. Studies of that kind would have very great value in determining when it was necessary to introduce reinforcement, and the manner in which arching took place. It was true that in the particular case in question, where the coal was removed and was replaced by a much weaker packing, the conditions were not entirely analogous; nevertheless the facts were of great interest, and especially the manner in which the pressure was distributed on to the various parts and shifted about according to the degree of compression.

Had the experimental results so far obtained given any indication of

the limiting depth of working in coal mines in Great Britain? Some years ago 4,000 feet was given as the probable limit for coal mines in comparatively soft rocks. Had the Authors any suggestions to make regarding the future possibilities of mining from that point of view, because that would have a very strong bearing upon British coal-resources?

With reference to the instruments used it occurred to him that the piezo-electric system would be very convenient for such observations, seeing that no immediate inspection was required with the instrument and no line of sight was necessary. The only disadvantage appeared to be the necessity for delicate electroscopes or elaborate amplifiers in order to obtain the required records.

Another aspect of the matter which was not mentioned in the Paper was the relation between the surface-subsidence and the manner in which the pressure differed from the static load. Zones of separation were referred to in relation to subsidence, and the latter was likely to become of greater importance in the future, in view of the fact that it caused considerable surface damage; had the Authors any suggestions regarding the relation between subsidence and the variation of the pressure from the static load? The total subsidence was bound more or less to correspond to the space created below, but some more definite indications would be of interest. Professor Karl von Terzaghi, M. Inst. C.E., had given a good deal of attention to the distribution of pressure around circular tunnels, and as there was undoubtedly similar or analogous behaviour in the lines of force around any kind of opening, it would be of great interest if the Authors could continue their research in that direction.

Mr. H. M. Morgans remarked that the dynamometer which was put into the centre of the pack to measure the compression had a rod which came out through the pack, with a scale on it which was read by a microscope. What was the order of movement of that rod? That had a bearing on the accuracy with which the pressure could be estimated.

The Authors discussed the changes in the pressure on the packs from the point of view of the packs, without special reference to the changes in pressure due to roof-measures. Various roof-strata might exist in different parts of the workings, and there might be joints, cleavage-planes, or dislocations, or there might be water in the ground. It was therefore very difficult to foretell the behaviour of the roof-measures, and he was not sure that such experiments as were under discussion were going to lead to results of any great value.

While extraction proceeded in a seam there might be a dome-shaped dislocation in the roof over the working area which the Authors called the "detached zone"; the pressure of that severed ground was felt at once on any packs or supports put in. That was followed by a movement of the upper measures, which widened as it went towards the surface so that the subsidence at the surface overreached the working face by a distance known as the "draw." When another cut was made, another primary dislocation

resulted, followed by another secondary disturbance. Some of the variations of pressure at the face was due to the changes which were taking place in the roof; there was first of all an abutment-pressure on the working face, then the pressure due to the weight of the detached ground, and then the pressure coming from the movement of the whole of the measures. He suggested, therefore, that it was not safe to consider what happened to the pack in compression only from the conditions of the pack itself, and that what was happening in the superincumbent measures had also to be taken into account.

Mr. J. S. Wilson observed that the Authors were to be congratulated on having made experiments under conditions which were bound to have been extremely difficult. He had had some experience of taking measurements in a tunnel near the working face, and of the damage miners or navvies did to delicate instruments. Professor J. A. S. Ritson had referred to the great depth of mines in South Africa; Mr. Wilson had been in one of those mines, and he had been rather astonished to find that at a depth of about 6,000 feet there was a chamber, apparently entirely unsupported, hewn out of the rock, that appeared to be nearly as big as the lecture-theatre at the Institution. That suggested that there was great strength in the rock, but there was a limit to the pressure which such rocks would carry when deprived of lateral support. In connexion with that point the experiments made by Professors F. D. Adam and E. G. Coker, M. Inst. C.E., were of interest.[1] They had prepared a number of little marble cylinders about 1½ inch long and just under 1 inch in diameter, and had fitted very accurately around each cylinder a mild-steel casing; they had then submitted that cylinder to very high axial pressure and had squeezed the marble down so much that although the mild-steel casing held it together, the power to hold the marble in place was not sufficient to keep it from spreading, and some cylinders increased in diameter by 25 per cent. After being distorted to that extent, the casing had been removed and the marble had been tested for crushing strength, and it had hardly lost any of its strength. Although even at such great depths as occurred in South African mines, the pressure was not so great as had been used in those experiments, it appeared to be getting near the point when rocks began to flow. It had been found in those experiments that if the temperature were raised slightly, the pressure to cause flow in the marble was not so great.

Mr. W. H. Evans said that attention had been drawn to the form of the load curve shown in *Fig. 7* (p. 347), which fell off at one stage. It might be thought that the reduction of load on the pack after maximum load, which was measured in several tests, would have produced some change in the rate of compression of the material. That procedure had been investigated in the laboratory with a 400-ton testing machine. The test was made on a pack measuring 10 feet by 5 feet in plan and 4 feet high,

[1] American Journal of Science, vol. 29 (1910), p. 465.

which was subjected to a load of 300 tons; that was to say, to an average pressure of about 100 lb. per square inch. The pack was compressed by about 25 per cent. The load was then reduced slowly and the recovery in height was observed. The recovery was very slow in the first stages, but it was more rapid as the load fell to lower values, and the total recovery was about 10 millimetres, or a little over 1 per cent. of the height of the pack at that stage. It seemed likely that the same form of relationship would hold for still higher initial loads. The recovery of height of a pack when the load fell from 50 to 25 per cent. of the initial load was about 0·01 inch, and was not measurable by the ordinary convergence recorder. In field-tests, however, reduction of load maintained over a considerable period of time would be offset by the concurrent yield in the pack under the remaining load.

*** **Professor Douglas Hay** observed that during the past few years increasing attention had been paid by mining engineers to the relatively high accident-rate due to falls of roof at the coal face. Much work had been carried out by the Safety in Mines Research Board in conjunction with the Support of Roof Committee of the Institution of Mining Engineers, and a great deal of material had been published. Those investigations had taken place in all districts, but so far few definite conclusions had been reached; in fact, the large amount of material already published was rather confusing. It was especially desirable that conclusions arrived at should be presented in a form which rendered them of practical service to colliery officials, so that the lessons derived might be employed in practice. The conclusions arrived at in the Paper could be studied with much benefit, although the Authors would probably be the first to admit that much remained to be done before the wide range of conditions encountered in British coal mines could be completely covered.

In spite of all the knowledge that had been obtained in recent years (some of which merely emphasized the value of traditional methods), it was the experience of mining engineers that the correct way of dealing with any given conditions of seam and roof could rarely be predicated, and in opening up a new seam practical experience was the only satisfactory way of ascertaining the correct method of supporting the roof, and of determining the disposition and strength of the packs, which eventually took the full weight of the superincumbent strata. It was common experience that before success was attained numerous changes of method might be necessary, and unfortunately even after a satisfactory method was adopted conditions of roof and floor and of the seam itself frequently changed in a distance of a few yards, necessitating the revision of the methods already adopted. It was impossible to see into the roof, and quite often a change occurred in the nature of what the Authors termed the

*** This and the succeeding contribution were submitted in writing.—Sec. Inst. C.E.

"detached zone," so that the first warning of trouble was the collapse of the face, the fall revealing the change in the strata.

The Authors had covered the principal factors involved in the consideration of what method of support should be adopted, although perhaps they had not sufficiently emphasized the importance of the spacing between packs in the waste; or, in other words, the number of points of support, and the strength of those points at which the packing system offered resistance to the roof. To that might be added the angle which the line of face bore to the natural cleavage in the rocks above. In many cases the overlying strata immediately adjacent to the coal seam broke up readily, and, falling between the packs, formed itself into a type of support which might have a determining influence. In one case he had in mind, a strong rock roof many yards thick in places practically lay directly on the coal seam, but in a few yards the bottom of the rock rose 20 feet above the seam, leaving in between a varying thickness of weak shales. In that case the width and distance apart of the packs had to be consistently varied in order to obtain proper control, and a uniform system was impossible.

It would appear that the method of support had to achieve at least three things: firstly, the control of the main roof to the surface at such a rate of lowering as not to cause undue or too rapid breaking of the rocks of the detached zone; secondly, the control of the rate at which the rocks of the detached zone itself settled and broke up; and thirdly, to control the immediate conditions under which the men were working at the coal face, and to ensure that large or small fragments of rock would not break away from the immediate roof and cause injuries. He could see no easy scientific method of dealing with that problem, and he felt that experience, quick judgement, and scientific training were the principal human factors on which depended any hope of reducing the accident-rate. In stating that he was not attempting to decry in any way the Authors' contribution, as it was essential to have full knowledge of the way rocks behaved under stress, and the way in which such stress operated, if methods were to improve.

Major H. M. Hudspeth had difficulty in agreeing with all the Author's conclusions, and in particular with the suggestion that by determining the height of the detached zone it would be possible to calculate the required strength of the supports. The load on a face prop was, as they stated, affected by the hardness of the roof and/or of the floor, which incidentally varied in practice. Again, the height of the detached zone was a function of the nature of the vertical sequence of the strata and also of the width of the working; indeed, it varied across the width, being less, for example, at a solid side. That changes occurred in the nature of the vertical sequence was well known. Other factors affecting the problem were the rate of face advance and the quality of the support afforded by the packing. How did the Authors propose to meet the effect of variations in those factors? Where wooden props were used a 4-inch, or perhaps prefer-

ably a 5-inch, diameter prop would be used in practice in a seam 4 feet thick. The strength of such props might be used as a measure of the resistance desired.

The conception of a detached zone was based, he believed, upon the fact that the measured loads on the supports represented the weight of the strata for a certain height above them. If the support were more rigid, however, it carried a greater load. Was the height of the detached zone then greater? The two factors were, in fact, inter-related, and the nature of the support determined the theoretical height of the detached zone.

The Authors, in reply, desired to express their appreciation of the welcome given to the Paper. The research was still in progress and it was hoped that an opportunity might occur to bring the subject again before The Institution when it had been further developed. Strata control was one of the many problems which engaged the attention of the mining engineer; Mr. Wentworth-Sheilds' remark that the research work of civil and mining engineers on "earth-pressures" was complementary and did not necessarily overlap, applied to other phases of research work, including, for example, haulage problems.

The research on strata control was being developed from two points of view:—

(*a*) To provide means to prevent falls of ground under present working conditions.

(*b*) To develop methods whereby the strata were maintained in good condition, in order to render falls less likely to occur.

The first aspect was the one which led to immediate results, as it depended upon modifications of permanent and temporary supports. The second aspect was more fundamental and far-reaching, and involved laboratory research into the physical properties of rocks, coupled with underground experiments. The two points of view were interconnected and the Paper covered both of them.

An essential condition in strata-control problems was to avoid high local loads in the regions of the working faces and roadways, and to offer greater resistance in parts where fracturing of the strata was unimportant. That was a wide subject and it offered many complications; for instance, it might be necessary to preserve a contiguous seam, either above or below, in its normal state for future working.

The behaviour of packs was referred to by Mr. Wentworth-Sheilds and Professor Ritson. It was shown by the experiment described on p. 348 that when three dynamometers were set across a pack the centre one recorded a load $2\frac{1}{4}$ times that of the two side instruments. That pointed to lack of constraint in the pack-walls as the probable cause. It was often an advantage at the roadside because it reduced the shear forces, and to some extent prevented the formation of concentric fractures over the pack and parallel to the axis of the road. On the other hand, if a strong wall

were built on the side of the pack remote from the roadway it might cause a fracture in the roof and permit it to fall, thus offering resistance to the lateral spreading of the pack.

The effect on roof conditions by providing a certain amount of yield in roadsides was very clearly seen in narrow roads driven in the bord-and-pillar system of working, where a heading driven parallel to the cleat of the coal (the wall), which had weak sides, invariably had better roof conditions than one at right angles to the cleat (the bord), where the sides were strong [1].

It was suggested that too close an analogy could not be drawn between the behaviour of an underground pack and the loads sustained in civil engineering practice. Underground packs were bound to endure the load if they were sufficiently wide to avoid collapse and if the roof and floor were strong enough, because of the constrained circumstances under which they were built. They became stronger as they were further compressed under an increasing load. A pack might suffer as much as 60 per cent. consolidation, and the material became nearly as hard and compact as the original rock. In that respect Mr. Wilson's remarks were of interest.

The Authors were in agreement that the effective resistance of a pack was offered by the filling and not the walls, which served the purpose of preventing the filling from spreading and developed the initial resistance during the early stages of compression. The success of packs made with very friable material restrained by wire netting confirmed that view. In normal practice the packs were extended as the face advanced, at a distance of about 12 feet behind it. The roof had subsided to some extent, depending on the thickness of the seam and in some cases as much as 15 inches, before the pack was made. Where pack walls were built, the walls offered resistance until the subsiding roof effectively exerted pressure on the filled interior, when, as shown in *Figs. 6* and *7* (pp. 346 and 347), pack-compression continued rapidly with a very gradual building-up of resistance during the earlier period. In the extreme case of a wire-netting pack built with friable material that material had to take the load forthwith. The experiment described by Mr. Evans was of interest. As he remarked, the reduction of load and its effect on the height of the pack would always be offset by the yield in the pack under the remaining load, and would therefore be negligible.

With regard to the point raised by Dr. Chatley, piezo-electric crystals could not be employed in the dynamometers for two reasons: (1) the difficulty of providing adequate insulation to prevent leakage of the minute currents involved; and (2) because they would require the use of flameproof amplifying apparatus, which would be too bulky and cumbersome for the purpose.

[1] Fourth Progress Report of the Support of Workings in Mines Committee. Trans. Inst. Min. E., vol. xci (1935–36), p. 349.

In reply to Mr. H. M. Morgans, the total movement of the rod connected to the bent steel strip in the dynamometer was 0·03 inch and the micrometer microscope read to 0·0001 inch, or 1 part in 300 of the maximum load.

Sir Henry Walker's remarks were very greatly appreciated. The inauguration of research on falls of ground and haulage problems by the Safety in Mines Research Board was largely due to his concern for the safety of underground workmen and for a reduction in the accident-risks.

It was very disquieting that accident-rates from falls of roof and sides did not show a marked improvement in recent years. One of the principal explanations was that the period during which the researches had been carried out had coincided with the fundamental changes brought about by the rapid introduction of machinery. The quantity of coal cut by machinery in 1928 was 61 million tons, or 26 per cent. of the output, and the figures increased to 137 million tons and 57 per cent. in 1937, the greater part of that output being won by longwall coal-cutters. The change from a slow rate of advance of the face under hand-getting conditions to one five or six times as great with coal-cutting machines, and the more impulsive loading of the roof, described in the Paper, by deep undercuts at great speeds, demanded different systems of working and more efficient methods of support; those were matters which were not always considered.

It had been shown that the function of the props was not to support large masses of strata, but to prevent the collapse of what might be termed the immediate strata, and of the detached slabs of stone which might constitute minor falls. That was confirmed during investigations of accidents from falls of roof, when it was very often found that no further fall had followed that which had caused the accident.

It would be realized that the longwall coal-cutter, although a highly specialized piece of apparatus, was a very clumsy machine to drag along a coal face. It introduced very serious roof-control problems, mainly because of the violence of the operation of undercutting and the inability to provide adequate support to the roof above the undercut coal until it was exposed by the miner several hours later.

The remarks on detached zones and the design of face-supports on a definite system made by Sir Henry Walker, Major Hudspeth, Professors Ritson and Hay, and Mr. Wilson, called attention to a very important problem. In workings where the roof was properly controlled, with an absence of face-fractures, with reasonable amount of subsidence between the coal face and waste, and where periodic weights did not occur, the height of the detached zone could be determined. The upper limit of that zone was at the first definitely strong bed in the roof strata which was able to span the distance between the solid coal and the position behind the face at which the permanent supports offered effective resistance to general subsidence. The measurement of prop-loads made it possible to follow engineering practice in calculating the strength of supports, the

bearing area between the support, the roof, and the floor, and the distances apart of supports. An adequate factor of safety should be provided, having regard to possible changes in the strata which might occur above and below the seam. Those strata-changes where not unexpected and did not present difficulties in ordinary circumstances. At present personal judgement would be applied in supporting the roof in faulted or difficult areas.

There were, however, many mines where the roof was not under proper control and where the conditions for calculating the strengths of supports were unfavourable. In those cases, which were either set up by undercutting under unfavourable natural conditions, or were due to a bad system of working, the roof on the working faces was fractured and dislocated, the subsidence between the face and waste was very excessive, and periodical weights occurred when the strata were fractured much higher than the horizon where the detached zone should be found under good working conditions. The loads on the supports might then increase until either the supports bent or broke, or the roof or floor failed, resulting in the roof collapsing.

The research on strata control was being developed to find ways of keeping the strata in good condition so as to render falls less likely to occur, but it would not be possible at any stage to say that the problem could be considered to be solved, because new methods were continually being introduced which brought in new sources of danger.

Production and research should proceed concurrently, and the pause in the investigations suggested by Professor Hay could not be contemplated. The results of research were embodied in scientific publications, and in addition booklets and pamphlets on practical subjects were issued by the Safety in Mines Research Board, written in popular language and well illustrated, for wide distribution amongst workmen. Addresses on roof-control problems were frequently given in mining villages in every coalfield, which were well discussed by the audiences, but it had to be admitted that much remained to be done before the results of research could be said to be effectively applied in practice.

Major Hudspeth raised the question of rigid supports, and suggested that the more rigid a support the greater the load it would carry. He also asked if the height of the detached zone would be increased by using more rigid supports. It was not considered practicable to maintain the roof-beds above the working face rigidly in their original position; indeed, they were moved before being exposed at the working face. Where the strata were bedded, slight separation occurred on bedding planes and a support set on the working face carried the initial load, due to the thickness of the lowest bed, which increased later as the strata subsided. If the system of support of the workings were correct, that slight subsidence of the roof was advantageous, as it relieved the immediate roof-beds of load from the beds above. It followed that more rigid supports might prevent

some bed-separation, and might therefore carry a greater load. When those supports were withdrawn at the edge of the goaf, the roof would probably break more freely and to a greater height at the first break. With less rigid supports, the first break in the goaf would not be so thorough in regard to height, but the higher beds would break in further stages as the face advanced. It did not follow that the height of the detached zone was increased by using more rigid supports, but it meant that the supports were carrying more of the strata within the detached zone. The strata within the detached zone were not devoid of self-supporting power. The system of support designed on loads expected from strata up to the detached zone would adequately meet any normal loading.

The conception that the loads measured by dynamometers represented the weight of the strata for a certain height above them, appeared to be permissible, and in practice it was confirmed by the behaviour of the roof on longwall faces, and also in broken lifts in bord-and-pillar workings. Subsidence of the roof in a working place did not necessarily imply excessive pressure, as the strata involved might be only a few feet of immediate roof; on the other hand, supports might be withstanding very considerable loads when practically no movement was recorded.

Steel had not made very rapid strides in replacing wood for supports on the working face, but it had the advantage of uniformity in strength, and its use was increasing. When steel props were set systematically and with lids of equal thickness, they offered equal resistances and permitted uniform roof-movement in favourable comparison with wood props, which varied very considerably in strength and which set up unnecessary strains in the roof from irregular subsidence.

The conception of an arch and its abutment-pressures had been raised by Professor Ritson, Dr. Chatley and Mr. Morgan. Although that was a very interesting problem it was felt that it should not be included in the Paper because more investigation was required, particularly regarding pressures in advance of the working face. No measurements had yet been taken in the centre of a waste between two parallel packs to determine the load transmitted by the overlying strata through fallen debris, as compared with the loads on the packs. Pack-loads in roads near to, and parallel to, coal-rib sides had shown that they were much smaller than the loads quoted in the Paper for the same depth, and no initial peak loads were recorded, as was to be expected [1]. The position appeared to be that the roof strata bent downwards fairly rapidly for a limited height and a short distance from the coal face towards the waste, causing a low initial pressure (increasing in amount) on the packs, which were offering resistance to the movement; the pressure continued to increase, until the packs were consolidated and the whole of the strata to the surface, or nearly so, had subsided by an amount about equal to the reduction in height of the packs. The solid goaf advanced in a line parallel to the coal face, usually

[1] Annual Report for 1937 of the Safety in Mines Research Board, p. 80.

at a distance of from 50 to 150 yards, and the subsiding strata in front of it had a curvature opposite to that at the coal face, with a much greater radius because the rate of movement was slower. Whilst the extraction of the seam was the initial cause of subsidence, it seemed probable that the surface draw, which was usually in advance of the coal face, and other phenomena met with on the surface, were directly connected with the line of the advancing solid goaf.

Dr. Chatley referred to the limiting depth of working. The researches of the Safety in Mines Research Board were directly concerned with underground mining conditions and safety, and the Authors were unable to suggest any limiting depth as a result of their work. The investigations made up to the present from the roof-control point of view, had not pointed to that depth having been approached, but other considerations, such as winding in shafts, and temperatures, were involved, and they might become deciding factors.

Professor Ritson's remarks on deep mining and the weathering of rocks were very valuable. In the introduction to the Paper the suggestion was made that strata movements involved other mining factors, such as haulage, the production of coal dust, and ventilation. With regard to accumulations of firedamp, ventilation was one of the subjects kept in mind when introducing changes in both temporary and permanent supports. The result to be aimed at was to maintain control to achieve regular and uniform subsidence both on working faces and over the wastes, in order to reduce the tendency to form dangerous cavities in the higher broken strata which might contain firedamp. The erratic collapse of a stratum above the waste was always a source of danger in that respect.

Measuring instruments would be used more extensively underground in future, and from the Authors' experience no difficulties were anticipated owing to interference or clumsiness by the miner, a possibility inferred by Mr. Wilson. In fact, many miners had been very helpful in reading instruments and making useful suggestions.

Further investigations were required on somewhat similar lines on working faces to attempt to avoid making the fractures which were being formed at present in the roof over the solid coal; that was to say, before it was presented to the miner in his working place. It was hoped that the dynamometers described in the Paper, used in conjunction with convergence-recorders, would be a valuable aid in that direction.

*** The Correspondence on the foregoing Paper will be published in the Institution Journal for October 1939.—Sec. Inst. C.E.

Paper No. 5153.

"'Gunite' Lining for the Channel Conveying the Water-Supply of Lashkar City, Gwalior, from the Reservoir to the Filters."

By Armell Richard Pollard, B.A., M. Inst. C.E., and Dhunjibhoy Cursetjee Rustomjee Baxter, B.Sc. (Eng.), B.E., Assoc. M. Inst. C.E.

(Ordered by the Council to be published with written discussion.)[1]

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Introduction.

Lashkar city, the capital of Gwalior State in Central India, derives its water supply from the Tigra reservoir, 9 miles to the west. The reservoir has an available capacity of 4,300 million cubic feet, and also provides water for some industrial concerns and for a certain amount of irrigation. According to the census of 1930 the population of Lashkar city and its suburbs was about 127,000.

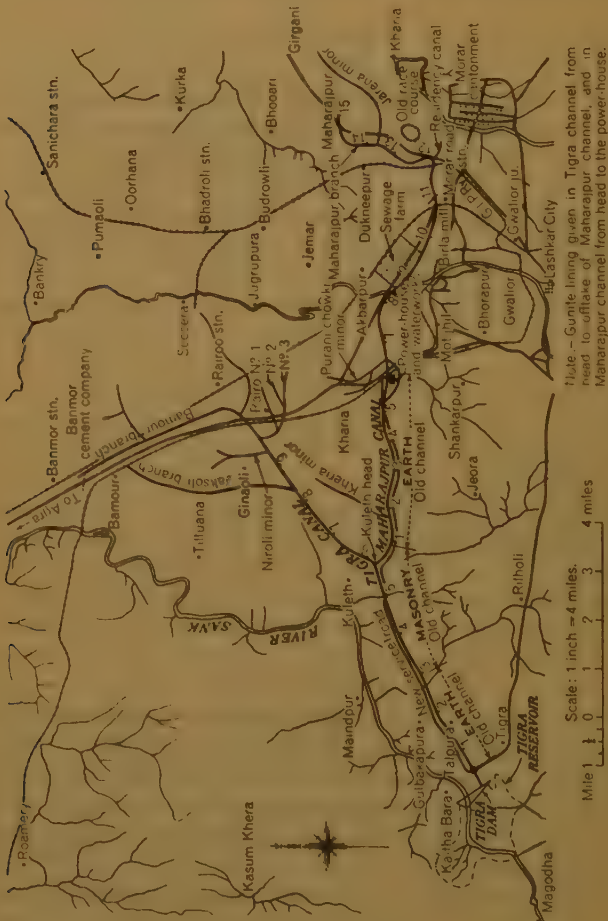
Water is conveyed from the reservoir to the filters in an open channel 1½ miles long (*Fig. 1*, p. 370). The "Gunited" length of channel consists of two parts, the first 5½ miles to Kuleth being the main channel, after which the water passes down the Maharajpur branch to the filters about miles farther on.

The channel that has been lined runs in sidelong ground throughout.

[1] Correspondence on this Paper can be accepted until the 15th April, 1939, and will be published in the Institution Journal for October, 1939.—Sec. Inst. C.E.

The first 3 miles are in sandy and *usar* soil (*usar* is the local name for soils containing a high percentage of salt, useless for cultivation and very treacherous in water), the original channel being of the usual kind in earth. For the next $2\frac{1}{2}$ miles the country is of fissured rock, strewn with boulders, with a thin covering of inferior soil. Here the channel consists of masonry

FIG. 1.



MAP OF CHANNEL-SYSTEM FED FROM TIGRA RESERVOIR.

side-walls with a masonry floor, generally of stone slabs (for which Gwalior is famous) on lime concrete. The side walls have a batter inside the channel of $\frac{1}{2}$ to 1. The first $\frac{1}{2}$ mile of the Maharajpur channel is also of masonry, and the remainder is in earth, the soil being a light and porous loam, with *muram* in places.

The cost of the reservoir and the combined water-supply and drainage

schemes in Lashkar was about Rs.7,337,000, and the cost of the channels before the gunite lining was laid was Rs.1,178,000. The total initial cost of the scheme was therefore over Rs.8,400,000 (£630,000 at par).

It was known that the losses in the channel between the reservoir and the filters were high, but the matter only became urgent in 1929 when the waterworks were nearing completion, and it was realized that the city's water-supply would be jeopardized unless the channel were made reasonably staunch. Gaugings and calculations showed the losses to be from 76 to 78 per cent. of the quantity of water issued from the sluice at the reservoir. In February, 1934, another set of gaugings gave the losses as 59 per cent., although Rs.191,000 had been spent on repairs in the meantime.

A Committee was appointed early in 1933 to examine the question. Its report, presented early in 1934, suggested either the provision of a pipe-line from the reservoir to the filters, or the lining of the channel in the lengths where the leakage was worst. These lengths aggregated about 5½ miles. The lining was recommended rather than the pipes, because the carrying-capacity of the channel would be much greater than that of a pipe of comparable cost, and losses of irrigation-water during the irrigation-season (November to April) would be eliminated. The type of lining favoured was "Gunite," not only because slabs of cement-concrete and stone had failed, owing to the difficulty of stopping the joints between the slabs, but also because "Gunite" had been successfully used in India for several purposes, though not, as far as was known, for lining irrigation channels. In the United States, however, "Gunite" is frequently resorted to for this purpose.

It was ultimately decided to line the whole 11½ miles with "Gunite." It was possible to do this without any increase in cost, because the section of the old earth channels, with a total length of about 8 miles, was much larger than the section of the "Gunite"-lined channel carrying the same discharges. In the original proposals the section of the lined lengths of earth channel was kept the same as the old section. The discharge of the channel is 100 cusecs at the reservoir and 45 cusecs at the waterworks. The sections of the "Gunite"-lined channel at the two places are:—

	Bed-width.	Depth of water.	Side slopes.	Bed slope.	Discharge.
At the reservoir .	*5·5 feet	4 feet	2/3	1/4,000	100–115 cusecs.
	*7·5 ,,	4 ,,	1/1		
At the waterworks	6 ,,	2·75 feet	1/1	1/5,000	45 ,,

* The channel has two sections. The original section has a 7·5-foot bed-width and 1/1 side slopes, whilst the later one, with 5·5-foot bed-width and 2/3 side slopes, was adopted on account of frequent cracks (see p. 380). The discharges of both the sections are about the same.

The discharges of the "Gunite"-lined channels were calculated by Manning's formula, using a coefficient of 0·017. This is on the high side, and 0·011 would probably have been nearer the mark, but in view of the great difficulty of altering the channel after the lining was finished, the discharge-calculations were purposely made conservative.

The total area of the "Gunite" is about 900,000 square feet.

Description of Lining.

For the channel from the Tigra reservoir to the Gwalior waterworks, the lining applied consists of a layer of "Gunite" 2 inches thick, with cement to sand in the proportion of 1 part to 3 parts by volume. For practical convenience one 1-cwt. bag of cement was assumed to contain $1\frac{1}{4}$ cubic foot. The lining is reinforced with "B.R.C." fabric, made of $\frac{3}{16}$-inch steel wire in 6-inch squares, electrically welded at the corners. The cement was supplied by a local firm, and the sand was obtained from the Sank river.

The top of the "Gunite" lining was kept about 3 inches above the full-supply level of the channel, and was surmounted by grouted stone pitching 1 foot thick laid at the same slope as the gunite, extending to a vertical height of 1 foot 3 inches. The object of this pitching is to prevent damage to the top of the lining and to stop erosion of the earth.

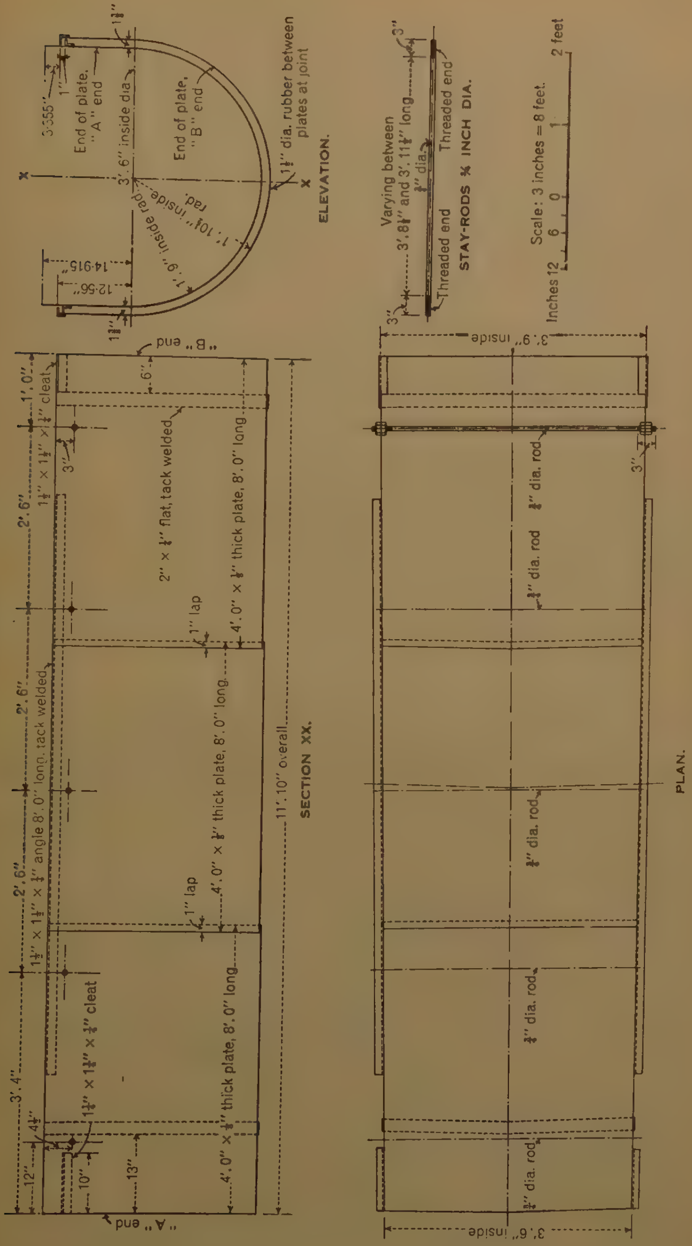
Where the channel is in masonry there is no special top to the lining, which has only been taken to the prescribed 3 inches above the full-supply level.

The combined estimate of the costs of the lining and the service-road was about Rs.1,200,000 (£90,000), of which Rs.150,000 to Rs.200,000 (£11,000 to £15,000) was for the road. The work was executed well within this amount.

Problems of Construction.

The application of the "Gunite" lining presented peculiar problems, because the supply for the waterworks and sewage-farm, and for certain industrial concerns in and around Gwalior had to be maintained without interruption during the entire period of construction. There were no communications in the tract through which the channels passed, and to make the work possible a service road had to be built. Further (and this proved to be the most troublesome part of the work), a labour force had to be kept always available for the earthwork that was needed to reduce the section of the old channel in earth to the correct section for the "Gunite."

Figs. 2.



STEEL FLUME-UNIT.

Maintenance of the Water-Supply.

To enable the lining work to proceed it was necessary to isolate lengths of channel, diverting the supply through a flume. This flume was made of ⅛-inch steel plate in units each 11 feet 10 inches long (*Figs. 2*), the cross section being a semicircle surmounted by vertical sides about 1 foot high. The diameter of the semicircle was 3 feet 9 inches at one end, tapering to 3 feet 6 inches at the other; this enabled successive lengths to be fitted together, leaving a joint 1⅜ inch wide, which was filled with a piece of 1½-inch-diameter rubber packing. The joints were tightened by driving four iron wedges (two on either side) between angle-iron cleats 6 inches long at the ends of each length of flume. The rubber packing was compressed when the wedges were driven home, forming an absolutely watertight joint. Each piece of flume was kept in shape by five ⅝-inch-diameter tie-rods, with threaded ends on which hexagonal nuts were fitted. To guard against buckling, angle-irons 8 feet long were welded to the upper edges of each length. When designing the flumes the dimensions were regulated so that each length could be handled by men, as the water-logged condition of the country and the absence of communications precluded the use of any tackle.

The flume structure was designed by Mr. A. J. Moore, General Manager of Messrs. John Fleming & Company, Ltd. (who carried out the "Gunite" lining work), as initially this firm had the pumping contract.

The flume was designed to carry a discharge of 24 cusecs with a bed-slope of 1 in 350, but in practice it was found convenient to adopt a slope of about 1 in 1,000, corresponding to a discharge of approximately 15 cusecs.

The original intention was to carry the flume on one of the channel banks, supporting it on saddles constructed of angle-iron, and to raise the supply from the channel into the flume by two or three high-speed pumps (according to the requirements), each of which was capable of discharging 3,000 gallons per minute (8 cusecs). Each pump had its own diesel engine, and was mounted on a small steel barge, which could be manipulated in the channel at normal depths. In the beginning, however, it was not possible to use the pumps because all three engines arrived with their crank-cases broken. To enable work to proceed, the flumes were laid outside the channel and the flow passed through by gravity. The existing banks generally provided enough freeboard upstream to allow for the requisite heading-up, and were patrolled continuously to stop leaks and prevent breaches.

The lining was begun from the waterworks end of the channel, because, since there was no leakage from the "Gunite"-lined channel, the flumes had only to pass the quantity needed. Had the work started from Tigra

reservoir the flumes and pumps would have had to be of impracticable dimensions.

Work started on the 8th November, 1934, with a small diversion 450 feet long, and the lengths of subsequent diversions ranged up to 2,600 feet ; the latter diversion was near the reservoir, where the masonry sides of the channel permitted considerable heading-up.

The flumes were laid either on the ground, in trenches cut in the ground, or in trenches cut in the outer slopes of the banks, depending on the ground-levels. The channel crossed most *nalas* (streams) by masonry aqueducts, and at such places the flumes were carried either on temporary wooden trestles or dry stone pillars. They were taken into the channel through cuts in the banks, or, where the sides were of masonry, through openings made in the masonry. Gaps in the lining caused by the ends of the flume-diversions were filled up subsequently. The gaps were isolated by small dams made with sacks of earth. The part of the channel to be lined when a flume diversion came into action was isolated by dams at either end made of sacks filled with earth.

The pumps were repaired and were brought into use on the 29th July, 1935, for the last two diversions before the Kuleth head, where there is a high masonry aqueduct 650 feet long, and thereafter a small section of masonry channel (6 feet bed width) in steep sidelong, rocky ground.

As stated on p. 370, the channel for 2½ miles upstream of the Kuleth head is built of masonry, the bed-width being 9 feet. Much of the channel is built high above the ground, and supports for flumes for gravity flow through the diversions would have been very expensive, so the pumps were again used, the flumes and saddles being supported by steel beams resting on the masonry side-walls.

When there was still nearly ½ mile of the masonry channel left to be lined, and a further 3 miles of earth channel beyond that, all three pumping engines broke down within the space of 3 days. The ground was well above the bed of the channel and was very rocky, so no flume diversions for gravity flow could be made. The bed-width of the channel being 9 feet and the outside diameter of the flumes 3 feet 9 inches, it was possible to lay the flumes on the bed against one side-wall and to line more than half the bed and the other side. The flumes were then moved across to the other side and the lining completed.

Fortunately the ground became undulating about 1,000 feet upstream of the place where the breakdown occurred, and it was possible to continue the old system of gravitational diversions outside the channel.

Preparation of the Channel for Lining.

Where the channel was in earth the preparations for the lining consisted of six stages, viz. :

(*a*) Isolation by diverting the flow through the flume.

(*b*) Dewatering.
(*c*) Removal of slush, etc.
(*d*) Reduction of section by filling in earth to the profile of the "Gunite" lining.
(*e*) Dressing new earthwork to profile.
(*f*) Placing reinforcement in position.

Dewatering was effected by making cuts in the bank, or in the side-walls of aqueducts where these were conveniently situated. The ends of diversions were sometimes fixed so as to include in the length of the diversion sites suitable for these cuts. On one or two occasions, where the channel ran in rock and the flumes were laid inside the channel, water had to be bailed out.

As soon as the channel was clear of water, slush and weeds had to be removed; on the sides the depth was from 3 to 6 inches, while in the bed the thickness was often from 9 inches to 1 foot.

The earthwork for reducing the section was begun immediately sufficient space to work in became available. This part of the operations was done by coolie labour, all earth being carried in baskets, so that careful organization was needed. The work was generally divided into lengths of about 100 feet. The earth was dumped in layers by the carriers, who were followed by men with hand rammers, who again were followed by water-carriers. As soon as one section was finished the next was taken in hand, all the carriers walking over the completed section, to help consolidation. The work was carried on thus until the top was about 9 inches higher than the designed top of the "Gunite" lining. A gang of coolies specially trained for dressing the earth then stepped in and trimmed the work to the correct profile. In this they were guided by templates that had been placed in position before the earth filling began, at intervals varying from 50 to 100 feet on straights and closer on curves.

A good surface was obtained by keeping the earth profile wet, using garden syringes. Experience showed that if the earth were not thus sprinkled it dried up and cracked, caving away in lumps where there was much clay in the soil, and becoming so friable as to be unfit for the "Gunite" where the soil was sandy.

The original design provided for a boulder filling immediately behind the "Gunite," but this proposal was discarded as soon as the work began as it presented great practical difficulties. The lining was therefore laid directly on the new earth backing.

The reinforcement was specially imported in sheets, which, for convenience in shipping and handling, were 14 feet by 6 feet and 16 feet by 7 feet. The lengths of the profile ranged from 15 to 21 feet. At first the top of the lining was tapered off after giving the top bar of the reinforcement a minimum covering of 1 inch, which entailed keeping the reinforcement from 3½ to 4 inches below the top of the lining. This was not in itself

unsatisfactory, but made it troublesome to lay the grouted pitching afterwards. In the final stages of the work the top edge was finished square, which was more satisfactory in every way, as it avoided the tendency to levelop cracks that was observed in the tops of the tapered lining, gave a neater appearance, and made it much easier to lay the grouted pitching. The fabric was bent at the junctions of the bed and sides. Sheets had to overlap from 6 to 9 inches along the length of the channel and from 3 to 6 inches around the profile. The joints were bound with wire at intervals of 18 inches.

Where the channel was in masonry the preparation of the surface for the lining consisted in a thorough cleaning by scraping and scrubbing with wire brushes. Joints were scraped out and dowel-pins driven in at suitable ntervals. The reinforcing fabric was tied to these pins.

Laying and Completing the Lining.

When the fabric had been laid the remainder of the work was carried ut in four stages :

(*a*) Shooting the "Gunite" lining.
(*b*) Curing the lining.
(*c*) Letting water into the newly-lined section.
(*d*) Laying the grouted pitching.

When laying the lining the procedure conformed closely to the general pecifications for "Gunite" of the Cement Gun Company, Inc. A few ninor changes were made to suit local conditions. The lining was "shot" ither in two layers of $\frac{3}{4}$ inch and $1\frac{1}{4}$ inch, or in one layer of 2 inches. n the beginning a flash coat was applied, but it was later discarded ecause it made no real difference in the finish of the work and retarded rogress.

In the intense heat and great dryness of Gwalior during the summer, the roper curing of the lining was of primary importance. Fresh "Gunite" as protected from the heat of the sun by covering it with sacking which as sprinkled with water when the lining was 2 or 3 hours old. After 4 hours water was admitted into the newly-lined length to a depth of inches, and was splashed on the sides by men.

As soon as the new lining had hardened enough to bear the weight of to $1\frac{1}{2}$ feet of water (generally after 48 hours, although this interval was ortened in certain cases, notably where the channel was in masonry), ater to this depth was allowed to flow. The sides were then covered by cking placed with its lower edge dipping in the water, and kept wet by en for 14 days in the earlier stages of the work, and later, in the hot

weather, for 21 days. The admission of water to depths of 1 to $1\frac{1}{2}$ feet established regular flow, as these depths corresponded to the discharges generally passed through the flumes.

Laying the grouted pitching was the final stage of the work. Before this could be placed the earth backing had to be raised to the full height This earthwork was allowed to pass through one monsoon in the 5 miles of channel immediately up-stream of the waterworks before the pitching was laid. It was not possible to follow this procedure near the Tigra reservoir as the period of construction would have been unduly prolonged. The result was that the pitching near the reservoir settled considerably and had to be reset in many places. The pitching consists of square stones, which were available close by and could be quarried in a way that left little dressing to do. The stone was laid in horizontal courses, generally two or three in number, carefully bedded in lime mortar.

Communications.

It has been mentioned above (p. 372) that there were no communica-tions in the tract through which the channel passes. For the constructio a road 12 miles long was needed between Tigra reservoir and the water-works (*Fig. 1*, p. 370). Between miles 12 and 6, where the lining was started the country was water-logged, and the banks of the channel were too narrow to allow any machinery to be taken along them. To enable the lining to be laid within the contract period the channel between the waterworks and the Kuleth head had to be finished between November, 1934, and July, 1935. For this the road between these two points had to be con-structed in such a way that the plant needed for the "Gunite" could move along the channel up-stream from the waterworks whenever the progress of the "Gunite" work required it. The decision to build the road was not arrived at until the contract for the "Gunite" lining had been made so it had not been possible to do anything beforehand.

The difficulty was overcome by assuming that the road would be made and building the bridge over the first *nala* upstream of the waterworks before any formal orders were issued. As soon as the orders were received all the bridges up to the Kuleth head were put in hand almost simul-taneously, care being taken that those just upstream of the place reached by the lining were always ready before they were wanted. The channel bank carrying the road was widened almost throughout to the minimum required for the 8-foot strip of macadam (about 11 feet). This widening was done on the outer slope, because the flow in the channel washed away a considerable percentage of any earth placed in the water and the dirty water caused complications at the waterworks. The bank-widenin

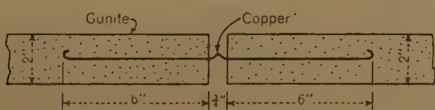
afforded useful work for the labour employed on the back-filling when no illing was being done.

The road between Tigra reservoir and the Kuleth head presented no lifficulty, as no lining work was in progress here while the road was being built. For 2 miles from the reservoir there was an old partially-made road, and it was only necessary to finish it. The country towards the Kuleth head was rocky, and it was not necessary to take the road along any of the channel-bank. This part of the road was ready in plenty of time.

Special Points in Connexion with the Lining.

The Kuleth aqueduct (see p. 375), had very weak side-walls. In he rains of 1932 a length of about 25 feet had collapsed, and the water-upply was cut off for a day or two. To avoid any further collapse the ops of the lining were braced by reinforced-concrete ties 3 inches wide by inches deep spaced about 10 feet apart. The ties were bound to the ining by two ½-inch-diameter bars at each end, penetrating 2 feet down nto the lining.

Fig 3.



The original intention was to have no expansion-joints, as advices from merica stated that none were built into "Gunite" lining, either for the urpose of joining-up the work of two consecutive days or for permitting ontraction and expansion, and that the lining was successful nevertheless. his does not necessarily mean that cracks do not occur, but that any hat may chance to develop are unimportant.

Experience on the Tigra channels showed that there is a tendency to rack at the ends of aqueducts, where there are changes in the shape of he channel, and at places where the nature of the backing changes (for xample, from masonry to earth). These cracks were not important, asmuch as the leakage through them was negligible, but since it was vident that they were preventable, expansion-joints were, during the ter stages of the work, provided at both ends of aqueducts. The joints ere formed of copper strips as shown in *Fig. 3*, the sides of the strip eing curved to provide the maximum adhesion between the "Gunite" ot before putting copper in position and that shot after. The expansion-

space between the ends of the "Gunite" was first filled up with a board whilst shooting, and, when the newly-shot "Gunite" had set, by a mastic compound. The joints achieved their object, no cracks appearing where they were made.

Such cracks as were found appeared during the winter, closing with the approach of the hot weather; the portions of the lining applied while the average temperature was low showed less tendency to develop cracks than those laid in hot weather, and the lining laid on an earth backing cracked more than that applied to masonry.

In the last stages of the work, when the 3 miles of earth channel near the Tigra reservoir were being lined, cracks appeared at short intervals of between 5 and 10 feet before the lining was 3 weeks old. It was thought that this might be due to the action of the earth backing, which shrunk on drying, or to the inclination of the side slopes (45 degrees) being too steep for the 5-foot height of earth behind them. The backing was accordingly kept wet for a fortnight, and the side-slopes were flattened to 1 vertical in $1\frac{1}{2}$ horizontal. Wetting the earth backing had no effect, and flattening the side-slopes very little. Washing the sand made some improvement, but did not stop the cracks altogether, so that some other explanation had to be sought. The majority of the cracks extended only to the water-level in the channel, but some went right round the profile, and were the more perplexing because the sand, water, and cement were the same as had been used from the beginning.

No entirely satisfactory reasons for this cracking have been found, but one cause may be the nature of the earth backing, which contained a substantially higher proportion of sand than the backing downstream. This deprived the "Gunite" of a portion of its water. The action was intensified by the nature of the sand, which consisted largely of *kunker* nodules and grains, porous to a considerable degree, some of which were loosely coherent and broke down when wetted, resulting in an increase of fine material. The rate of absorption from the "Gunite" remained just below the critical point with the more clayey earths downstream, but exceeded it in the 3 miles of channel near the reservoir. The sand-washing was not sufficient to reduce the rate of absorption below the critical point, as there was a very definite limit of wetness, beyond which the sand clogged the box of the cement-gun.

By the time the foregoing became known the lining was nearly finished, and in view of the fineness of most of the cracks it was only necessary to repair them. This was easily and successfully done by cleaning with wire brushes, damping, and brushing-in cement grout, repeating the operation where necessary.

About $2\frac{1}{2}$ miles down-stream from the Tigra reservoir, a small drainage-area of $3\frac{1}{4}$ acres on the right was intercepted. For many years it had been efficiently drained by a small cut through the ridge that carried storm-water to a river nearby. In August, 1936, there was a very heavy fall of

rain on this drainage-area, which caused the water outside the channel to rise to about 6 inches below the top of the right bank, as the cut could not discharge quickly enough. With an ordinary earth channel this would have had no effect beyond causing seepage into the channel, but with the impervious "Gunite" lining the unbalanced lifting force was about 40 lb. per square foot. This lifted the lining along a length of 600 feet. Where the lifting did not exceed 3 inches no damage resulted, but in one place the lining in the bed was raised $1\frac{1}{2}$ foot, and burst. In places the sides caved in. The caving on the left bank was caused by the presence of an old cement-concrete slab laid in a former attempt to stop the leakage and buried in the earth backing. This slab prevented the storm-water from finding its way out to the left, after passing under the lining.

A culvert with 10 square feet of waterway was built under the channel at this place, and the lining was removed where necessary and relaid with 3-inch mesh reinforcement. Quartz sand was brought from 12 miles away. These precautions, and the saturation of the ground in the monsoon, avoided a recurrence of the cracks mentioned above.

The consolidation of the earth backing for the "Gunite" lining was described on p. 376. After the monsoons of 1935 and 1936 this backing was carefully examined for settlement, which could be easily detected where present by tapping the lining lightly with a wooden rod. Hollows were filled by pouring earth and water behind the lining, and working the earth lown with bamboo strips. Very little filling was wanted where the backing had passed through two monsoons. It is anticipated that after three monsoons the settlement of the backing will be complete.

The lining should not be overtopped near high banks, as the grouted pitching is not watertight, and the bank might be washed away. In the channel up-stream of the waterworks screens are fixed in the channel to prevent weeds from entering the filters. These screens get clogged, and water is often headed-up above the top of the "Gunite." To obviate the langer mentioned above the grouted pitching has been covered with ement plaster where necessary.

Weeds abound in the Tigra reservoir, whence they pass into the channel, where they grow with great vigour between August and November. The "Gunite"-lined channel, with its small section, might easily become choked unless constant vigilance were exercised. A method of dealing with these weeds, other than the simple one of employing gangs of men especially for he work, has yet to be evolved.

Losses from the $11\frac{1}{2}$ miles of "Gunite"-lined channel are negligible. Advantage was taken of the opportunity afforded by the existence of a pecial construction division to repair certain parts of the old channel not ncluded in the lining scheme, adopting less expensive measures than "Gunite." The main item was staunching the leaks in a masonry conduit hat carries the water from the reservoir-sluice underneath the outfall from he waste weir and into the channel.

One of the Authors, as Chief Engineer (Irrigation) of the Gwalior Government, was responsible for the design, the methods employed, and the preliminary organization of the work, and the other was the Executive Engineer directly in charge of the construction.

The Paper is accompanied by three sheets of drawings, from some of which the Figures in the text have been prepared, and by five photographs.

Paper No. 5177.

"An Experimental Study of the Voussoir Arch."

By Professor ALFRED JOHN SUTTON PIPPARD, M.B.E., D.Sc., M. Inst. C.E., and RONALD JOSEPH ASHBY, M.Sc. (Eng.), Stud. Inst. C.E.

(Ordered by the Council to be published with written discussion.) [1]

TABLE OF CONTENTS.

INTRODUCTION.

In an earlier Paper [2] an account was given of the first stage of a research nto the behaviour of the voussoir arch. That Paper dealt only with the nechanics of the problem, and described the results of experiments upon a small model-arch in which the voussoirs were made of mild steel. The present Paper describes an extension of this work to a larger structure built of materials of a more practical nature.

The earlier work was largely concerned with the behaviour of an arch which was pinned at the supports, although sufficient attention was given to the arch supported on skewbacks to show the general nature of the behaviour of such a structure when carrying a concentrated point-load in addition to the dead weight of filling.

In those experiments the voussoirs had no jointing material between them, so that the usual assumption made in design, that mortar is incapable of resisting any tensile stress, was exactly satisfied. With this condition it was established that, provided the correct linear arch fell everywhere

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[1] Correspondence on this Paper can be accepted until the 15th April, 1939, and will be published in the Institution Journal for October, 1939.—SEC. INST. C.E.

[2] A. J. S. Pippard, E. Tranter, and L. Chitty, "The Mechanics of the Voussoir Arch". Journal Inst. C.E., vol. 4 (1936–37), p. 281 (December 1936); *discussion*, vol. 6 (1936–37), p. 5 (June 1937).

within the middle third of the arch-ring, the structure behaved exactly like an arch-rib and could be analysed by the standard methods applicable to such structures.

An increase in the point-load finally produced a condition in which adjacent voussoirs were in contact only at the extreme edges and a succession of virtual pin-joints were formed, which reduced the problem to comparatively simple statical terms. Failure occurred when sufficient of these pins formed to transform the structure into an unstable mechanism.

Between the two extremes of an arch-rib and a pinned structure there was an intermediate stage in which the bearing surface between adjacent voussoirs extended over a diminishing depth of the joint, but owing to the nature of the material used in the voussoirs this transition-stage was not very marked.

If a jointing material is used in the construction of an arch the adhesion between this material and the voussoirs, or the nature of the material itself, may enable the joint to offer some resistance to tensile stress. Also, the material may be such that it will fail in compression before the virtual pin can form at the extreme edge of the joint. These factors, combined with the possibility of failure of the voussoirs themselves, made it desirable to continue work with different materials from those used for a study of the mechanics of the problem.

The general effects of the presence of jointing material of the type referred to will be to increase the magnitude of the point-load under which the structure behaves as an arch-rib, since the linear arch can fall outside the middle third without causing the opening of a joint, and to lower the magnitude of the load at which instability occurs, since the "pin" cannot reach the extreme edge of the joint.

If the material of which the voussoirs are made cannot withstand the compressive stresses set up, premature failure of the structure will occur.

Choice of a Linear Arch.

Before proceeding to an account of the experiments carried out to investigate these points, attention must be drawn to an expression used above. Reference was made to the "correct" linear arch, and this needs explanation.

In the generally accepted method of design it is assumed that, if a line of resistance can be drawn to lie wholly within the middle third of the arch-ring, the structure will be safe. Any number of such lines can be drawn, and the choice of a particular one is generally quite arbitrary. It does not appear to have been recognized that each of the possible lines which can be drawn is intimately associated with the conditions of fixity at the supports, and this fact needs emphasizing.

Suppose that *Fig. 1* represents a voussoir arch with fixed ends, which carries any system of external loads. Since it is assumed, as a basis of

design, that no tension shall be developed in the ring, the structure can be treated as an arch-rib, as already shown.

Such a rib has three redundant reactions which will be represented by the couple M_0, the vertical force V_0, and the horizontal force H_0, at the support B. The forces V_0 and H_0 act at the centre of the springing.

These three reactions can be replaced by a single force R acting at some point in the springing line, and this is the point through which the line of resistance passes. Since the structure is complete in itself if the end B is unsupported, the actions M_0, H_0, and V_0 may be assigned any arbitrary values without causing a collapse. For each set of values, however, there will be a different position of the resultant action R; that is, the line of resistance will start at a different point in each case.

Fig. 1.

These actions will, however, in general, cause component displacements of the end B which may be calculated by an application of the first theorem of Castigliano. Thus, if U denotes the total strain-energy of the structure due to the external system, then

$$\frac{\partial U}{\partial M_0} = \text{the angular rotation of section B in the direction of } M_0,$$

$$\frac{\partial U}{\partial H_0} = \text{the horizontal movement of section B in the direction of } H_0, \text{ and}$$

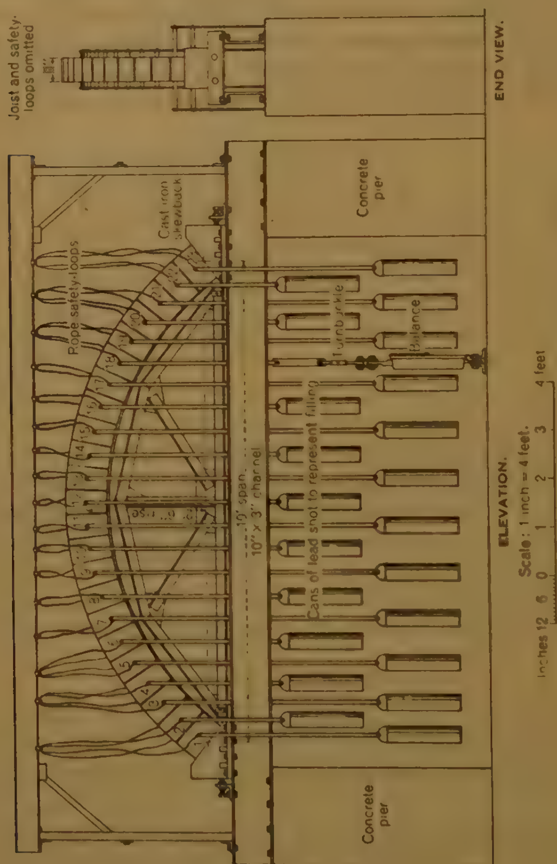
$$\frac{\partial U}{\partial V_0} = \text{the vertical movement of section B in the direction of } V_0.$$

For one particular set of values of M_0, H_0, and V_0 these movements will be zero; that is, there will be no displacement of the support. These values correspond to absolute fixity of the abutments, and the resulting linear arch is then theoretically correct for the assumed conditions of the ends. For any other set of values, however, the end B will be displaced and a different linear arch will be obtained. The arbitrary assumption of a particular linear arch thus tacitly assumes certain movements of the abutments, and it is therefore inconsistent to assume absolute fixity and at the same time to select a linear arch arbitrarily.

The Experimental Arch.

For the experiments described in this Paper it was desirable to build as large an arch as possible consistent with convenience of testing under laboratory conditions. A span of 10 feet and a rise of 2 feet 6 inches were therefore decided upon. The general arrangement of the arch ready for test is shown in *Figs. 2.*

Figs. 2.



General Arrangement of Experimental Arch.

Stone voussoirs would have been most suitable for the purpose, but the expense of dressing was prohibitive, and concrete was used instead. The voussoirs, shown in *Figs. 3*, were cast in steel moulds and were 10 inches deep and 6 inches by 5·8 inches at the mid-section. Two sets of voussoirs were made of different mixtures, the first set having pins inserted for strain measurements, as shown in *Figs. 3*. These pins were not needed, however,

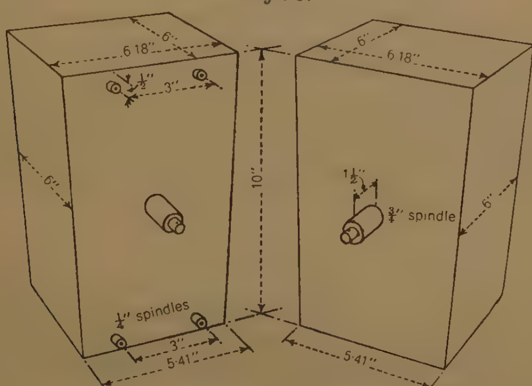
and since they were a source of weakness they were omitted in the second set of voussoirs.

The arch-piers were made of concrete and were bridged by two 10-inch by 3-inch rolled-steel channels rag-bolted into the concrete. These channels tied the piers and prevented spreading of the abutments of the arch. The skewbacks were made of cast iron and were bolted to the bridging channnels. A " frog " was left in the springing face of each skew-back, which was filled with concrete. This avoided a cast-iron to mortar joint, which might have caused uncertainties.

No attempt was made to measure the thrust of the arch in these experiments, since the evidence obtained in the earlier research was conclusive on this point.

The arch, when mortar-jointed, was formed of twenty-three voussoirs,

Figs. 3.



Concrete Voussoirs.

each joint being $\frac{1}{4}$ inch thick. In two series of experiments the voussoirs were in direct contact, and twenty-four were needed to complete the ring. The rise of the arch was then rather more than 2 feet 6 inches, but allowance for the variation was made in all calculations for this case.

The centering for erecting the arch was a steel frame made of structural sections, supporting a wooden former on which flats were cut to locate the voussoirs exactly. The centering was provided with adjustable screws which rested on the bridging channels. These screws enabled the centering to be accurately adjusted and readily struck.

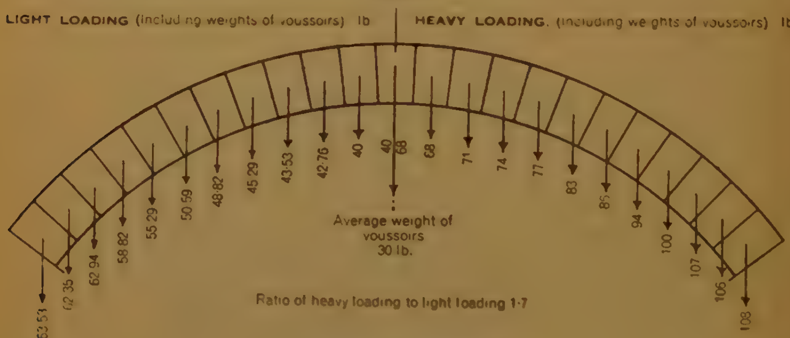
An A frame on each pier carried an overhead joist, to which each voussoir was attached by a loose rope sling. This was a precaution against damage in the event of a sudden collapse of the structure, and for extra safety the centering was not removed during tests, but was only lowered sufficiently for the purpose required.

The load representing the arch filling was provided by lead shot carried

in cans from the individual voussoirs. These cans were carried on steel pins cast in the voussoirs, and shown in *Figs. 3*. The majority of the tests were made under a light loading, the distribution of dead load being shown in *Fig. 4*. One test was, however, made with heavier loading [1].

The first set of voussoirs were made of rapid-hardening cement sand, and $\frac{1}{4}$-inch limestone chippings in the ratio by weight of 1 : $1\frac{1}{2}$: 3. These were found to spall rather easily, partly due to their composition and partly to the presence of the corner rods previously mentioned and shown in *Figs. 3*; a second set was therefore cast, from which the rods were omitted and in which the limestone chippings were replaced by granite chippings ranging between $\frac{1}{8}$ inch and $\frac{3}{8}$ inch. These will be distinguished as limestone and granite voussoirs respectively.

Fig. 4.



LOADING SYSTEMS.

Compression-test cubes were made of both materials and tests on these gave the following results :

The average compressive strength of limestone concrete at 28 days was 1,740 lb. per square inch.

The average compressive strength of granite concrete at 28 days was 6,700 lb. per square inch.

SCHEDULE OF TESTS.

Seven series of tests were made on arches built and loaded to the specifications shown in Table I (p. 389).

The non-hydraulic lime gives a mortar with practically no tensile strength, and it was used solely to form a bedding for the voussoirs. It is also weak in compression compared with cement mortar. The cement mortar was a mixture of rapid-hardening Portland cement and sand in the proportion 1 : 3 by weight.

[1] The light loading was estimated on a filling of 6 inches at the crown of the arch, the density being 70 lb. per cubic foot. The heavy loading was estimated on a filling of 12 inches at the crown, the density being 140 lb. per cubic foot.

TABLE I.

Series.	Voussoirs.	Jointing material.	Loading (see *Fig. 4*).
1	Limestone.	Non-hydraulic lime mortar.	Light.
2	,,	none	,,
3	,,	Rapid-hardening Portland cement mortar.	,,
4	Granite.	Non-hydraulic lime mortar.	,,
5	,,	none	,,
6	,,	Rapid-hardening Portland cement mortar.	,,
7	,,	,, ,,	Heavy.

DESCRIPTION OF TESTS.

The procedure adopted in building the arch was as follows. The centering was accurately erected by means of the four levelling screws, and the voussoirs rested on the flat surfaces previously mentioned. Small wooden wedges were placed between the voussoirs, so adjusted that the distances between them were equal. After the joints were made these pegs were removed. For the first tests these joints were hand-packed with a fairly dry mortar, but it was found on dismantling the arch after test that small voids were left in the mortar, and in subsequent tests all joints were grouted.

After an interval sufficient to allow the mortar to set, the centering was lowered so that the arch could settle under its own weight and assume its natural position. Observations showed that only very slight movements occurred during the first few hours after the centering was struck, and none at a later date when the dead load was applied.

For the tests of series 2 and 5, in which no mortar was used in the joints, an alteration to the centering was necessary since, due to shrinkage and possibly slight inaccuracies in the moulds, the voussoirs were not exactly true to their designed dimensions The frogs in the skewbacks were, for these two series of tests, filled with concrete and levelled with plaster of paris. The slight variations in the proportions of the arches without mortar from those of the other series were taken into account in the analysis of results.

In a certain number of cases it was possible to repair joints damaged during test without disturbing the remainder of the arch.

An important point to be determined from the experiments was the magnitude of the load which caused the first tension-crack to appear in a joint, and several methods to render this crack readily observable were tried. The first method used was to cement thin glass microscope-slips to the voussoirs, one such slip bridging each joint at the intrados and another at the extrados. This was abandoned, however, since in some cases the glass slipped on the cementing material. It was finally found

that small dabs of plaster of paris smeared across the joints where required formed excellent " tell-tales ", and showed very clearly the occurrence of the first crack. This method was therefore used for all subsequent tests.

After the dead loads had been applied to the arch the mortar was given sufficient time to set properly before the test was made.

Each test consisted in applying an extra load to one or other of the voussoirs by means of a turnbuckle attached to a spring balance, which was anchored by two hook-bolts to the flanges of a 10-inch by 6-inch steel floor-joist upon which the arch-piers were built, the balance having been previously calibrated in a 5-ton testing machine. The load was gradually increased by suitable increments while observers kept careful watch on the tell-tales. The normal observations made were the loads causing the appearance of the first tension-crack and subsequent cracks, and the positions of these failures.

The test was continued until complete failure occurred, usually by the development of a fourth " pin-point " causing the structure to become unstable, or in some cases by spalling of the voussoirs, or by slipping along a joint.

It was found that the load could be steadily increased to the value at which a fourth pin developed, when a sudden collapse occurred. The centering of the arch prevented a complete break-up, and on removing the point load it was generally found that the structure returned to its original position unless slipping between voussoirs had occurred.

Test-Results.

Twenty-six separate tests were carried out as described above, and in accordance with the schedule given on p. 388. The results of these tests are shown in Tables II to VIII, each Table relating to the arches in a particular series. For easy reference the tests were given serial numbers which are entered in the first column of each Table. The second column specifies the voussoir to which the concentrated load was applied. The three succeeding columns give the load which caused cracks to appear in the plaster tell-tales. The important value is that at which the first crack showed, since until this load is reached the structure can be treated as an arch-rib. The load causing complete failure of the structure, that is, the maximum load it was possible to apply, is entered in the next column. The positions of the joints which failed are given in the four following columns in order of appearance. The joints are specified by reference to the adjoining voussoirs ; for example, 8–9 e means that the joint between voussoirs 8 and 9 cracked at the extrados. Failure at the intrados is denoted by the addition of i to the joint-reference. The voussoir numbers are shown in *Figs. 2* (p. 386).

It should be remembered that the formation of four pins causes the arch to be transformed into a mechanism and failure occurs due to insta-

TABLE II.—RESULTS OF TESTS: SERIES 1.

No.	Load-point.	Loads: lb.				Points of failure.				Material failure.
		First crack.	Second crack.	Third crack.	Failure.	First crack.	Second crack.	Third crack.	Fourth crack.	
1	20	1,015	1,365	1,435	1,565	19–20 i	8– 9 e	23– 0 e	0– 1 i	C
2	19	562	1,062	1,112	1,252	18–19 i	8– 9 e	0– 1 i	23– 0 e	—
3	18	678	678	1,078	1,228	17–18 i	23– 0 e	8– 9 e	0– 1 i	—
4	17	660	930	1,180	1,250	16–17 i	0– 1 i	8– 9 e	22–23 e	—
5	16	583	783	883	1,333	8– 9 e	15–16 i	22–23 e	0– 1 i	—
6	15	585	955	955	1,405	8– 9 e	14–15 i	22–23 e	0– 1 i	Sp., Sl.

TABLE III.—RESULTS OF TESTS: SERIES 2.

No.	Load-point.	Loads: lb.				Points of failure.				Material failure.
		First crack.	Second crack.	Third crack.	Failure.	First crack.	Second crack.	Third crack.	Fourth crack.	
7	18	635	875	875	1,175	17–18 i	22–23 e	2– 3 i	8– 9 e	Sp.
8	17	547	977	1,077	1,177	16–17 i	2– 3 i	10–11 e	23– 0 e	Sp.
9	16	380	880	1,330	1,330	15–16 i	8– 9 e	0– 1 i	21–22 e	Sp.

TABLE IV.—RESULTS OF TESTS: SERIES 3.

No.	Load-point.	Loads: lb.				Points of failure.				Material failure.
		First crack.	Second crack.	Third crack.	Failure.	First crack.	Second crack.	Third crack.	Fourth crack.	
10	18	1,125	1,575	1,775	1,825	17–18 i	7– 8 e	23– 0 e	0– 1 i	Sp.
11	16	1,433	1,783	2,233	2,233	15–16 i	7– 8 e	22–23 e	0– 1 i	Sp.

TABLE V.—RESULTS OF TESTS: SERIES 4.

No.	Load-point.	Loads: lb.				Points of failure.				Material failure.
		First crack.	Second crack.	Third crack.	Failure.	First crack.	Second crack.	Third crack.	Fourth crack.	
12	20	995	1,415	1,415	1,465	19–20 i	9–10 e	23– 0 e	0– 1 i	—
13	19	272	672	872	1,252	18–19 i	9–10 e	23– 0 e	0– 1 i	—
14	18	528	1,028	1,128	1,228	17–18 i	0– 1 i	8– 9 e	23– 0 e	—
15	17	610	850	1,140	1,250	16–17 i	0–·1 i	7– 8 e	23– 0 e	—
16	16	433	783	1,003	1,403	15–16 i	7– 8 e	0– 1 i	23– 0 e	—
17	15	505	505	505	1,855	14–15 i	8– 9 e	8– e	23– 0 e	C.

TABLE VI.—RESULTS OF TESTS: SERIES 5.

No.	Load-point.	Loads: lb.				Points of failure.				Material failure.
		First crack.	Second crack.	Third crack.	Failure.	First crack.	Second crack.	Third crack.	Fourth crack.	
18	18	765	765	765	955	23– 0 e	17–18 i	10–11 e	0– 1 i	Sp., Sl.
19	16	133	133	1,103	1,203	23– 0 e	15–16 i	10–11 e	0– 1 i	Sp., Sl.

TABLE VII.—RESULTS OF TESTS: SERIES 6.

No.	Load-point.	Loads: lb.				Points of failure.				Material failure.
		First crack.	Second crack.	Third crack.	Failure.	First crack.	Second crack.	Third crack.	Fourth crack.	
20	20	1,565	1,965	2,335	2,235	19–20 e	8– 9 e	23– 0 e	0– 1 i	—
21	19	2,252	2,522	2,672	2,672	18–19 e	9–10 e	23– 0 e	1– 2 i	Sp.
22	18	1,525	2,075	2,075	2,075	17–18 e	9–10 e	22–23 e	1– 2 i	Sp.
23	17	980	2,560	2,820	2,820	16–17 e	8– 9 e	23– 0 e	0– 1 i	Sp.
24	16	1,463	2,683	2,683	2,683	15–16 e	7– 8 e	23– 0 e	0– 1 i	Sp.
25	15	1,565	3,165	over 3,360		14–15 e	7– 8 e	21–22 e	0– 1 i	Sp.

TABLE VIII.—RESULT OF TEST: SERIES 7.

Test number	
26	Load-point . . . . 18 First crack and instability failure, simultaneously at 3,545 lb. Points of failure . . . 23–0 e; 17–18 i; 7–8 e; 0–1 i. Spalling and slipping occurred at 17–18 e.

bility. A typical failure of this sort is shown in *Fig. 5*, one of the pins being shown in *Fig. 6*.

In all tests instability caused failure, but in some cases the failure occurred prematurely, either by crushing of the mortar (as shown in *Fig. 7*, facing p. 395), denoted by C in the last column of the Tables, by spalling of a voussoir (denoted by Sp.) or by slip occurring at a cracked joint (denoted by Sl.), as shown in *Fig. 8* (facing p. 395).

The results of the tests are shown in diagrammatic form in *Figs. 9* and *10* (p. 396). *Fig. 9* gives the tests on both limestone and granite voussoir arches when jointed in lime mortar, and *Fig. 10* those when cement mortar was used. Reference curves are plotted in each figure as follows. Curve A shows the concentrated load which must be applied to any voussoir to cause the linear arch to reach the middle-third point of any joint; that is, it is the load which would be considered to be the maximum allowable if the usual assumptions in design were correctly used. Curves B and C are similar to A, but show the loads which cause the linear arch to reach the middle-half and middle-three-quarter points respectively of any joint. Curve D gives the loads which cause the linear arch to touch the extrados of the arch-ring at any joint. All of these curves are obtained on the assumption that no cracking of a joint occurs; that is, that the arch can be treated as a rib. Curve E gives the maximum loads for which an arbitrary linear arch can be drawn wholly within the middle third of the arch ring. As shown on p. 385, this involves the tacit assumption that the abutments are free to move as required, and although this method is widely used it is incorrect if the abutments are rigid, as they are normally assumed to be. For the particular proportions of the experimental arch it is of interest to note that this curve coincides very closely with curve B, which shows the maximum loads keeping the linear arch within the middle-half. The middle-half rule has been advocated by various authors and, at any rate in the test-arch, its correct application would lead to the same results as the usual and incorrect application of the middle-third rule. Curve G gives the loads which would cause the arch to fail due to instability. The method of calculation is given in detail in the Appendix (pp. 402 *et seq.*). Curve F is similar to curve G, but is based on the assumption that the arch ring is 9 inches deep instead of its real value of 10 inches. This assumption was made as it was found in a number of the experiments

Fig. 5.

ARCH AT FAILURE: THE ARROWS SHOW JOINTS WHERE "PINS" FORMED.

Fig. 6.

NORMAL PINNING.

5177.

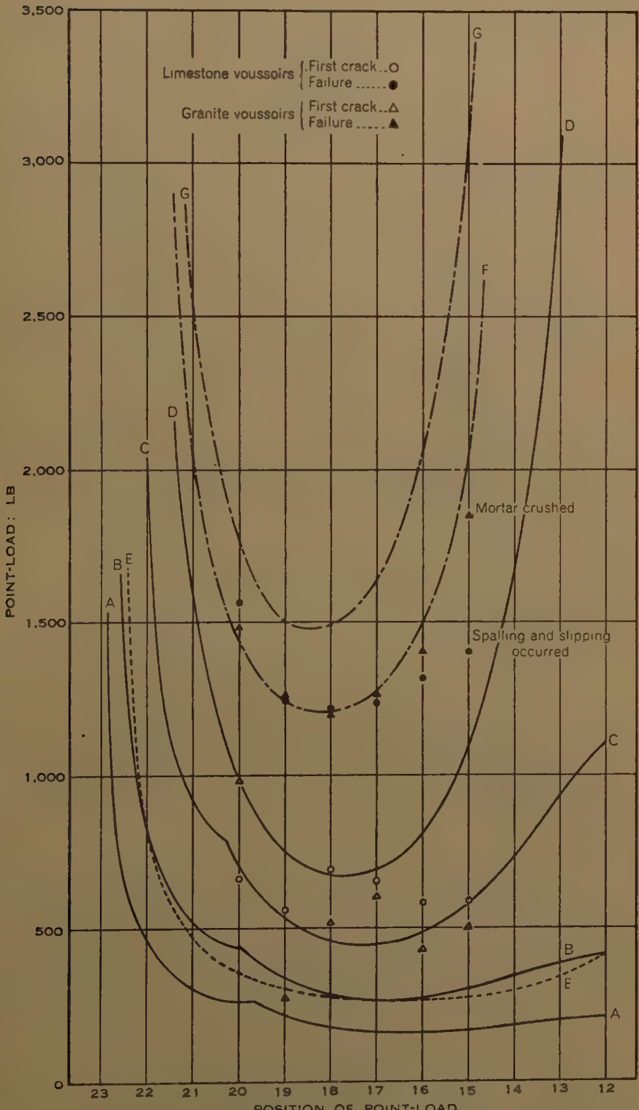
Fig. 7.

CRUSHING OF MORTAR.

Fig. 8.

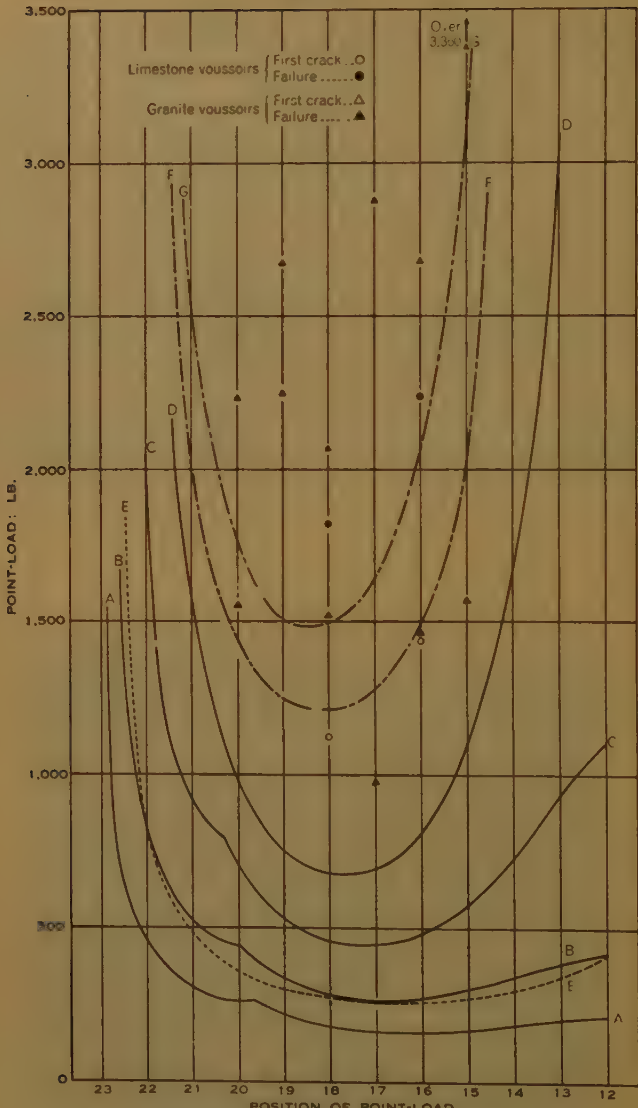
SPALLING AND SLIPPING.

Fig. 9.



TESTS WITH LIME-MORTAR JOINTS.

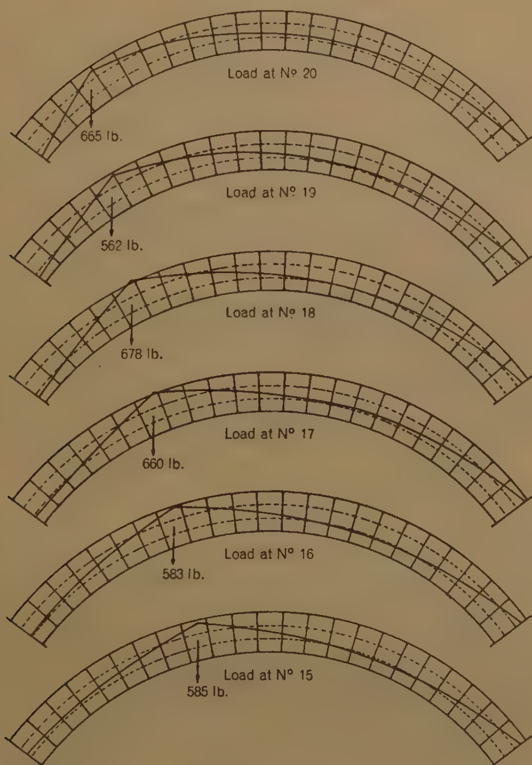
Fig. 10.



TESTS WITH CEMENT-MORTAR JOINTS.

with lime mortar that the tension-crack only extended to within $1\frac{1}{2}$ inch of the edge of the ring at failure. If a linear distribution of stress is assumed the centre of pressure is then $\frac{1}{2}$ inch from the edge. This is,

Figs. 11.



Lines of Resistance at the Appearance of the First Cracks.
(Limestone voussoirs, lime-mortar joints, light loading.)

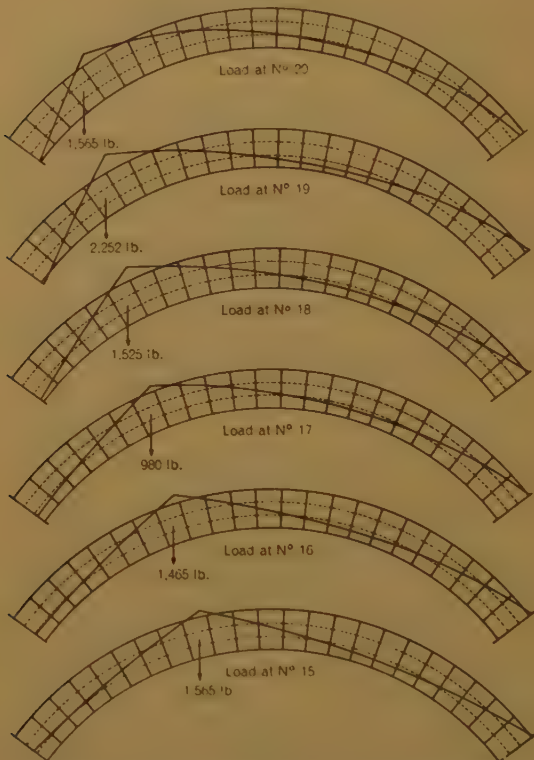
however, not a very satisfactory assumption, and the curve is only included since so many experimental points group around it.

Discussion of Results.

If attention is directed in the first place to *Fig. 9* (p. 395), relating to lime-mortar tests, it will be seen that only in one case did the first crack appear at a load below that of curve B. The majority of these tests did not show any sign of cracking until the linear arch had nearly reached the extrados of the ring. The ultimate failures were all below the values given by curve G and most of them were grouped closely around curve F. Those

giving low values of ultimate load failed by crushing of the mortar or spalling of the voussoirs, as indicated in *Fig. 9*. It should be noticed that the results of the tests are not dependent to any marked extent upon the material of the voussoirs, indicating that the behaviour of the arch depends

Fig. 12.



LINES OF RESISTANCE AT THE APPEARANCE OF THE FIRST CRACKS.
(Granite voussoirs, cement-mortar joints, light loading.)

primarily on the jointing material. This was expected to be the case, and except for a greater tendency to spall in the first limestone voussoirs (*Figs. 3*, p. 387), the expectation was borne out.

If *Fig. 10* (p. 396), relating to tests with cement-mortar joints, is now examined, it will be seen that in every case the linear arch came well outside the arch-ring before the first crack appeared, and in the majority of cases ultimate failing loads were much higher than those based on the stability calculation for unjointed voussoirs. These results indicate that cement-mortar joints have an appreciable strength in resisting tension, which could well be taken into account in design.

The results obtained are shown in a different way in *Figs. 11* and *12* (pp. 397–8), where a series of linear arches are shown for lime- and cement-mortar jointed arches respectively. These linear arches are drawn for the loads at which the first crack appeared, and show the very conservative nature of the middle-third rule. These linear arches were obtained by means of a strain-energy analysis, based on the assumption that there was no movement of the abutments.

These tests were all carried out for the light dead loading, since the point-loads required to cause failure under the heavy loading would have been inconveniently large. For comparison, however, one test was done with the heavy dead load, and the result is given in Table VIII (p. 394). The arch showed no sign of failure until a point-load of 3,545 lb. was applied, when it collapsed suddenly in a typical instability failure. The spalling was no worse than in previous tests. Under light loading the arch failed at 2,075 lb. applied to the same point (test 22). The ratio of the failing loads is 1·71, which is practically the ratio of the dead loading.

The tests made without any jointing material between voussoirs were very erratic, as might have been expected. The loads causing instability were much lower than calculated, due to spalling and slipping.

Conclusions.

The work described in the present Paper is supplementary to that previously carried out[1], and it is useful now to summarize the principal results of the complete research and to draw certain conclusions.

(1) A voussoir arch will, within certain limits of loading, behave as an elastic arch-rib. The end reactions and stresses in the arch can be calculated by the usual theory applicable to such ribs.

(2) The limit of loading for this treatment will be reached when tensile stresses develop which cannot be transmitted across a joint.

(3) If a joint is cracked at the intrados by tension due to the application of a point-load, there will be a progressive opening of this joint as the load is increased, and adjacent voussoirs will ultimately only make contact on a small part of their depth. These voussoirs are then virtually connected by a pin at the extrados. A further increase in the load will cause a second joint to crack at the extrados, leading ultimately to the formation of a second pin at the intrados. If the arch were pinned at the supports it would then become unstable and collapse. If, however, the arch has fixed ends, the load can be further increased until instability is caused by the formation of two more pins at or near the skewbacks.

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[1] Footnote (2), p. 383.

(4) It is customary in design to assume that the jointing material is incapable of resisting any tensile stress, and to limit loads to those which keep the linear arch within the middle-third core. It is generally assumed that if any linear arch can be drawn within this core the specified condition is satisfied. This is incorrect; there is only one correct linear arch consistent with the assumption of completely fixed ends. The use of any other linear arch tacitly assumes movements of one abutment relative to the other. The correct linear arch can be calculated by the usual methods (for example, by strain-energy analysis) as long as no crack has occurred at any joint.

(5) Although the lime mortar used as a jointing material in the tests had apparently no tensile strength, it did in fact develop sufficient tensile strength to prevent joint-failure until the linear arch was well outside the middle-third core.

(6) Cement-mortar joints developed sufficient tensile strength to prevent joint-cracking until the linear arch was well outside the arch-ring.

(7) After the first crack appeared in the lime-jointed arch, there was a big reserve of strength before failure occurred by instability. The load producing instability was less than that calculated on the assumption that no jointing material was present, as the lime crushed and caused the virtual pins to form at short distances from the edges of the arch-ring.

(8) The tensile strength of the cement-mortar joints not only delayed the appearance of the first crack, but also raised the ultimate loads to values considerably higher than those calculated for the unmortared structure. After the appearance of the first crack, it was usual for no other crack to appear until the arch suddenly failed by instability; that is, the last three cracks developed practically simultaneously.

(9) Spalling of the voussoirs caused premature instability in certain tests, but this danger can be reduced or eliminated by using suitable materials for the voussoirs.

(10) Slip only occurred in the tests as an accompaniment of crushing or spalling, and never as a distinct type of failure.

It is evident from these tests that, even when a weak mortar is used, the middle-third criterion is unduly pessimistic. This has been recognized by various authors, who have suggested that the middle-half core should be considered as the safe region for the linear arch. With good cement mortar, however, even this is very much on the safe side, and there is little doubt that no cracking of joints would occur if a wider margin still were adopted. It must be emphasized, in this connexion, that the development of tension at a single joint does not indicate that the structure is unsafe, since there is a very large margin of strength available before instability occurs.

The experimental arch was not, of course, subjected either to traffic conditions, which give repeated loads, or to weathering effects; it may therefore be argued that continual application of the load might cause a joint, subjected to tension, to open ultimately, and that this is undesirable, even if it is not dangerous.

If this argument is considered to be valid it is necessary to design so that the linear arch remains in the middle-third core, but if this is done the structure should be treated as an arch-rib and designed by the methods applicable to such members. The use of the arbitrary-linear-arch method is incorrect.

It is intended to carry out further experiments to determine the effect of repeated loading.

This research was done in the Civil Engineering Laboratories of the Imperial College of Science and Technology, and the thanks of the Authors are due both to the Department of Scientific and Industrial Research and to the Clothworkers' Company, whose respective research grants made it possible to carry out the work.

The Paper is accompanied by nine sheets of drawings and by four photographs, from which the Figures in the text and the half-tone page-plate have been prepared, and by the following Appendix.

[APPENDIX.

APPENDIX.

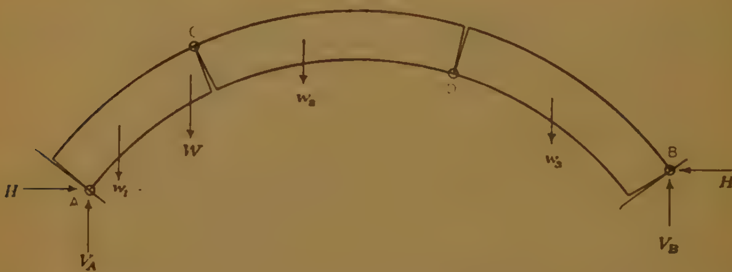
Calculation of Load to Produce Instability.

Instability of a voussoir arch is caused by the formation of four virtual pins, occurring alternately on the intrados and extrados. An example of an actual failure is shown in *Fig. 5* (facing p. 394), the arrows indicating the joints where the pins formed.

Fig. 13 represents an arch carrying a specified dead load system denoted by W_1, W_2, and W_3. The load W just causes the structure to assume a state of unstable equilibrium. The reactive forces are then V_A, V_B, and H.

In the first place it will be assumed that the positions of the four virtual pins A, B, C, and D are known. The section of the structure CDB is in equilibrium under the forces acting upon it, and these do not include the unknown load W. Hence, by taking moments of these forces about C and D, two equations in H, V_B, and the appropriate part of the dead-load system, are obtained. By eliminating V_B from these equations the value of H is determined absolutely.

Fig. 13.



If moments of the forces on the structure are taken about A, an equation is obtained which contains W, V_B, H, and the dead-load system; V_B can again be eliminated by using the moment equation about C, and since H has already been found, the required value of W is determined.

In an actual case the positions of two of the pins are not precisely known, and some trial and error is necessary. Certain general rules can, however, be given which reduce the uncertainty considerably. Thus, pin C always forms at the extrados of the joint adjacent to W lying between W and the crown of the arch; pin B always forms at the extrados of the springing farther away from W; pin A is on the intrados, either at the other springing or on a joint near it, and the position of pin D can be judged very nearly in most cases.

To obtain the value of W it is therefore necessary to make a few trials for different pin-positions. H is calculated from a consideration of the equilibrium of the section CB as before, but for a few (perhaps two or three) different positions of D. The position giving the lowest value of H will be the correct one, since this will give the lowest value of W whatever the position of A. The correct value for H having been found, a few alternative positions for A are taken and the values of W calculated as already described. The lowest value of W will be that producing instability.

The work is best done systematically, and the following method has been devised to enable the calculations to be made as easily as possible.

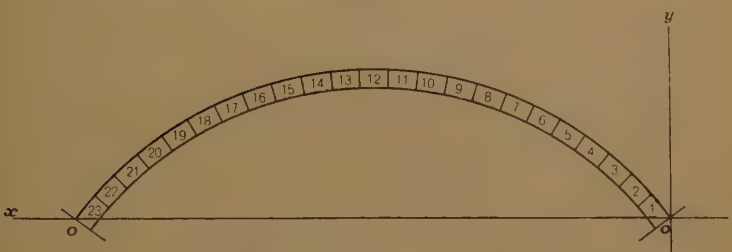
It is convenient to choose pin B as an origin with Ox as horizontal and Oy as vertical axes of reference, as shown in *Fig. 14.* Starting from the left-hand abutment the x and y co-ordinates of the extrados and intrados of each joint are tabulated (in this particular case in terms of the mean radius), as shown in Table IX (facing p. 404). The x co-ordinate of the centre of each voussoir is also entered in column 4 of the Table, the dead load applied to it in column 6, and the moment of this dead load about the axis through O in column 7. The sum of all dead loads to the right of a joint and the sum of their moments are then entered in columns 8 and 9.

As an example of the use of such a Table the calculations to determine the load which must be applied to voussoir No. 19 to cause the arch in *Fig. 14* to become unstable will be carried out in detail.

By taking moments about C for the part of the arch between C and B the following equation is obtained :

$$x_C V_B = y_C H + x_C \sum_{C}^{B} w - \sum_{C}^{B} wx.$$

Fig. 14.



LOAD-SYSTEM SHOWN IN TABLE IX.

Point C corresponds to 18–19 e, and on substituting numerical values from the Table this becomes

$$1{\cdot}329 V_B = 0{\cdot}272 H + (1{\cdot}329 \times 1501) - 886,$$

whence $$V_B = 0{\cdot}204 H + 835. \qquad (1)$$

For a first trial, pin D will be assumed to occur at 9–10 i, and the moment equation for the part of the arch between D and B about this point is

$$0{\cdot}622 V_B = 0{\cdot}345 H + (0{\cdot}622 \times 835) - 227,$$

whence $$V_B = 0{\cdot}554 H + 471. \qquad (2a)$$

Eliminating V_B between (1) and (2a) gives

$$H = 1040. \qquad (3a)$$

For a second trial, pin D will be chosen as 10–11 i;

then $$0{\cdot}700 V_B = 0{\cdot}357 H + (0{\cdot}700 \times 906) - 274,$$

whence $$V_B = 0{\cdot}511 H + 516. \qquad (2b)$$

From (1) and (2b)

$$H = 1042. \qquad (3b)$$

Similarly, the selection of 8–9 i as pin D gives the value

$$H = 1045 \qquad (3c)$$

The position 9–10 i is the correct one since it gives the lowest value for H.

Next, the selection of the left abutment (that is, 23–0 i) as the pin A, gives

$$1{\cdot}600V_B = -0{\cdot}027H + (1{\cdot}600 \times 0{\cdot}2016) - 1649 + 0{\cdot}247W,$$

whence

$$V_B = -0{\cdot}017H + 986 + 0{\cdot}155W \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (4)$$

Eliminating V_B between (1) and (4), and substituting for H from (3a), gives

$$W = 512.$$

The selection of 23–22 i as the pin A leads to the value

$$W = 534.$$

The correct pins are therefore, 0–1 e; 9–10 i; 18–19 e; and 23–0 i, and the corresponding values for the load and horizontal thrust are $W = 512$ lb. and $H = 1040$ lb. respectively.

TABLE IX.

Twenty-three voussoirs, 10 feet span, 2 feet 6 inches rise.
Depth of arch-ring 3·33 inches.

Col. (1)	Col. (2)	Col. (3)	Col. (4)	Col. (5)	Col. (6)	Cl. (7)	Col. (8)	Col. (9)
Reference-points.			x.	y.	Load, w.	Moment, wx.	Σw.	Σwx.
Int.	Mid.	Ext.						
23, 0			1·600	—0·027				
		23, 0	1·636	0			2,016	1,649
	23		1·593		108	172		
22, 23			1·550	0·035				
		22, 23	1·584	0·064			1,908	1,477
	22		1·540		106	163		
21, 22			1·496	0·091				
		21, 22	1·526				1,802	1,314
	21		1·481		107	159		
20, 21			1·437	0·144				
		20, 21	1·465	0·178			1,695	1,155
	20		1·419		100	142		
19, 20			1·369	0·191				
		19, 20	1·399	0·228			1,595	1,013
	19		1·353		94	127		
18, 19			1·307	0·233				
		18, 19	1·329	0·272			1,501	886
	18		1·283		86	110		
17, 18			1·237	0·270				
		17, 18	1·256	0·310			1,415	776
	17		1·210		83	100		
16, 17			1·165	0·301				
		16, 17	1·181	0·342			1,332	676
	16		1·135		77	88		
15, 16			1·090	0·326				
		15, 16	1·102	0·368			1,255	588
	15		1·057		74	78		
14, 15			1·013	0·345				
		14, 15	1·022	0·388			1,181	510
	14		0·978		71	69		
13, 14			0·936	0·357				
		13, 14	0·941	0·401			1,110	441
	13		0·898		68	61		
12, 13			0·857	0·364				
		12, 13	0·859	0·408			1,042	380

Col. (1)	Col. (2)	Col. (3)	Col. (4)	Col. (5)	Col. (6)	Col. (7)	Col. (8)	Col. (9)
Reference-points.			x.	y.	Load, w.	Moment, wx.	Σw.	Σwx.
Int.	Mid.	Ext.						
	12		0·818		68	56		
11, 12			0·778	0·364				
		11, 12	0·777	0·408			974	324
	11		0·737		68	50		
10, 11			0·700	0·357				
		10, 11	0·694	0·401			906	274
	10		0·657		71	47		
9, 10			0·622	0·345				
		9, 10	0·613	0·388			835	227
	9		0·578		74	43		
8, 9			0·545	0·326				
		8, 9	0·533	0·368			761	184
	8		0·501		77	39		
7, 8			0·471	0·301				
		7, 8	0·455	0·342			684	145
	7		0·425		83	35		
6, 7			0·398	0·270				
		6, 7	0·379	0·310			601	110
	6		0·353		86	30		
5, 6			0·328	0·233				
		5, 6	0·306	0·272			515	80
	5		0·283		94	27		
4, 5			0·267	0·191				
		4, 5	0·237	0·228			421	53
	4		0·217		100	22		
3, 4			0·199	0·144				
		3, 4	0·171	0·178			321	31
	3		0·154		107	16		
2, 3			0·140	0·091				
		2, 3	0·109	0·123			214	15
	2		0·096		106	10		
1, 2			0·085	0·035				
		1, 2	0·052	0·064			108	5
	1		0·043		108	5		
0, 1			0·036	—0·027				
		0, 1	0	0				

Paper No. 5180.

"Railway Track-Work for High Speeds."

By John Taylor Thompson, M.C., M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*)[1]

The Paper points out that since 1932 the mileage run at scheduled speeds of over 60 miles per hour in Great Britain has increased from 2,134 to 11,228, and in the United States of America from 2,022 to 37,412. Track-design is not subject to the same conditions as exist in most structural work owing to the obscure nature of the loads and stresses involved, and present standards have been evolved as a result of experience rather than as the outcome of theoretical design. The introduction of high speeds, however, increases the importance of analyzing, at least in a general way, the nature of the forces involved.

The subject is treated under the following headings: (1) Alignment and Cant; (2) Stresses in Track; and (3) Maintenance.

Under the heading "Alignment and Cant" the question of length of transition-curve and comfortable speed are treated in detail. For curves without cant the length of transition should be as given by Mr. W. H. Shortt, M. Inst. C.E.[2], namely $L = 0{\cdot}000724V^3/R$, where L denotes the length of the transition in chains, V the speed in miles per hour, and R the radius in chains. For curves with cant it is recommended that the cant should be gained at a rate of $1\frac{1}{2}$ inch per second, from which $l = \frac{E}{1{\cdot}5} \times \frac{V}{3{,}600} \times 5{,}280$ $= 0{\cdot}98\,EV$, or say $l = EV$, where l denotes the length of the transition in feet, E the cant in inches, and V the speed in miles per hour.

In dealing with comfortable speed it is pointed out that with any speed in excess of that for which the curve is canted the passenger will experience a sensation similar to that experienced on a longitudinal seat in an electric train when accelerating or braking. Braking tests therefore form an ideal basis on which to decide when the acceleration (radial acceleration in the case of curves) becomes noticeable and, at a higher figure, objectionable. It will be realized that in measuring comfort no absolute figure is possible, and that a range of varying degrees of comfort is all that can be expected.

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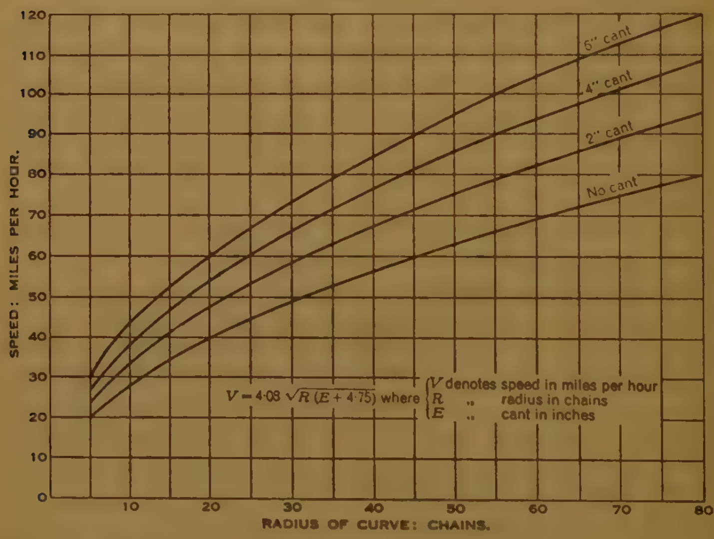
[1] Copies of the complete Paper can be obtained on loan from the Loan Library of The Institution; a limited number of copies is also available, for retention by members, on application to the Secretary.

[2] "A Practical Method for the Improvement of Existing Railway-Curves." Minutes of Proceedings Inst. C.E., vol. clxxvi (1908–9, Part II), p. 97.

In other words, a range of speed can be obtained varying from that at which the effect of curvature is not noticed to that at which the effect is just bearable. The evidence of braking tests indicates that an acceleration of 2 feet per second per second is not noticed and that one of $3\frac{1}{2}$ feet per second per second is bearable.

Adopting 2·6 as the comfortable average of these limits for the radial acceleration, then $v^2/r = 2{\cdot}6$, and the centrifugal force is $(W/g) \times 2{\cdot}6 = 0{\cdot}081\ W$. With this centrifugal force applied at the centre of gravity of the engine, say 5 feet 11 inches (71 inches) above rail-level, the resultant force will intersect the track 5·75 inches from the centre, giving a factor of safety against overturning of 5·13. This corresponds to a cant of

Fig. 1.



$\frac{59}{71} \times 5{\cdot}75 = 4{\cdot}78$ inches, or say $4\frac{3}{4}$ inches, as the permissible cant-deficiency dictated by the comfort of the passenger.

The speed in relation to cant and radius for equilibrium conditions may be expressed as $V = 4{\cdot}08\sqrt{RE}$, where V denotes the speed in miles per hour, R the radius in chains, and E the cant in inches. Introducing the amount of permissible cant deficiency as obtained above the comfortable speed on circular curves is found to be $V = 4{\cdot}08\sqrt{R(E + 4{\cdot}75)}$.

Fig. 1 gives the speeds for various radii for both canted and uncanted curves based on this formula. Based on the above reasoning, the amount of cant to be used in conjunction with the maximum high speed is given as $E = 0{\cdot}06\ \frac{V^2}{R} - 4{\cdot}75$. From *Fig. 1* it is clear that a comparatively large

increase of cant does not give a correspondingly large increase in permissible speed.

The action of a vehicle on a curved track is discussed at some length, and further experimental work is suggested to throw light on this most obscure form of track-loading.

Under the heading of "Stresses in Track," conclusions are drawn from American research on the subject, and it is stated that to increase the resistance of the track to depression attention should be directed to improving the rail-support, that is, sleeper, ballast and formation, rather than to increasing the weight of the rail.

One function of track-design is to distribute the wheel- and train-loads in such a way that they reach the earth with a pressure well within its bearing capacity. The ballast is the final means of effecting this distribution. Assuming the total weight of an engine to be 168 tons and the length 71 feet, the load per square foot on the formation is 0·2 ton, which is well within the capacity of even poor ground. This is the ideal to be aimed at, and either by sleeper-spacing or depth of ballast the load should reach the formation with the least possible variation of unit pressure. Variation of pressure produces uneven compression, uneven compression leads to depressions, depressions collect water, and water is the root of all evil in track maintenance. With 5 inches of ballast under the sleepers rather less than one-half of the formation receives the track-load when the sleepers are spaced at 2-foot 7-inch centres, whilst with 10 inches of ballast the pressure is distributed over the whole area, although with only a very slight pressure at the midway point between the sleepers.

In dealing with the subject of maintenance it is pointed out that minor defects, of only corresponding importance when low speeds are involved, become most important when high speeds are introduced. A mere $\frac{1}{4}$ inch in level in conjunction with $\frac{1}{2}$ inch in alignment have been known to play havoc in the kitchen- and the dining-car. Just as small defects upset the running, so small corrections are rewarded by greatly improved running. Drainage is stressed as one of the most important features of track-maintenance, and suggestions are made for dealing with wet places in the track. Following a review of the principal items of track-maintenance a form of track-inspection based on the award of marks for various classes of work is described.

The Paper concludes with a suggestion that the development of high-speed traffic seems to be a good reason for undertaking further experimental work on such questions as the lateral flange-thrust on curves.

The Paper is accompanied by one folding plate, seven illustrations in the text, and four photographs of instrument-records.

ENGINEERING RESEARCH.

THE REPORT OF THE FUEL RESEARCH BOARD FOR THE YEAR ENDED 31ST MARCH, 1938.[1]

Special attention is given to the work of the Coal Survey, which has now reached a stage justifying a general review of its progress and results. The survey was preceded by a period of some years devoted to discussion and planning of the organization, to a review of existing information concerning coal resources, and to the enlistment of the interests of those engaged in the industry. In 1922 the organization was put to work, and since that date Advisory Committees and Coal Survey Laboratories have been established successively in all the principal coalfields of the country. The staffs of the Coal Survey Laboratories work in close co-operation with mining and research workers in their districts, and in consultation with the Geological Survey. The first stage of the work is the survey of the coal seams as they exist below ground; for this purpose pillars are cut from the face of the seam at different points in the coalfield, and are transported to the laboratory, where their physical and chemical characteristics are investigated. The second stage consists of the examination of all the commercial grades of coal actually sold by the collieries; frequent tests on coal from the seams being surveyed are carried out on industrial-scale plant at the Fuel Research Station. Coal seams seldom remain constant in properties over wide areas, and whilst in some coalfields the changes encountered are irregular in both the amount and direction of change, in others marked directional trends are evident, though the rate of change may not be uniform. In several instances, notably in Northumberland and Durham and in South Wales, it has been possible to plot the analytical results for particular seams on maps of the coalfields, and from the points so obtained to produce a map in which the variations are accurately contoured. These maps show the total extent of the change in properties of the seam over the coalfield and the rate and direction of maximum change at any point. Where, as sometimes happens, a company is working coal in an area of rapid change, these maps have an important practical application in the methods of mining and marketing the coal.

The value of the Coal Survey is cumulative, and increases rapidly with the completeness of the information available; the stage has now been reached where the results are being utilized both to help the more rational development of the various seams and to indicate the distribution of coal

[1] H.M. Stationery Office, 1938. Price 4*s*.

with special characteristics. Complete surveys have been carried out on sixty-five seams, and work on over thirty others is in hand. Full and reliable data have been collected and co-ordinated on approximately 30,000 million tons of coal, representing about a quarter of the proved coal reserves of Great Britain, and a much larger proportion of those likely to be in active development in the near future. The work of the Survey provides an accurate chart of the coal resources of the country; it is effectively summarized in the Report, and references are given to the Survey Papers in which the details have been published.

The Report deals also with the many other activities of the Board. Progress has been made in methods of sampling and analysis, and in studies of the constitution and properties of coal. Many "commercial samples" of coal as marketed have been examined, and various problems of its preparation are under investigation. Results of tests on deterioration of coal in storage are discussed. Further work has been done on the "dust-proofing" of coal by the addition of small quantities of oil.

Experiments in connexion with the carbonization and gasification of coal have been directed primarily towards increasing the flexibility of the gas- and coke-making industries and widening the range of coals that can be used by them. It has been shown that, with slight modifications to the plant, weakly-caking coals can be used for making water-gas; further experiments are directed to the enrichment of the water-gas, and to increasing its hydrogen-content to make it more suitable for the catalytic synthesis of hydrocarbons. The earlier work on the hydrogenation of "low-temperature" tar has been extended to other tars, and experiments on the cracking of tars and hydrogenated oils have been begun.

The utilization of the hydrocarbons produced by catalytic synthesis from carbon monoxide and hydrogen has been studied; motor-spirit and high-grade diesel oil are obtainable, and the important possibility of producing lubricating oils by polymerization of the unsaturated constituents of the crude product is being investigated. Work on the direct hydrogenation of coal has been continued, the properties of catalysts and the effects of variations of pressure, temperature, and rate of hydrogen-input being examined.

Further improvements have been sought in the operation of pulverized-fuel burners; the investigation of cylinder-wear due to the use of pulverized fuel in internal-combustion engines has been continued, and work on a petrol-engine supplied with coal-dust in the air-intake is now being extended by tests on a two-cylinder diesel-engine of which one cylinder is being converted to powdered-fuel firing.

WORK IN PROGRESS AT THE LABORATORIES OF THE BRITISH ELECTRICAL AND ALLIED INDUSTRIES RESEARCH ASSOCIATION, DECEMBER, 1938.

In the Electrical Research Association's Auxiliary Laboratory at Perivale research is carried on in practically all branches of electrical engineering and its associated mechanical and physical problems. The following notes indicate briefly the nature of some of the more important work at present in progress.

Power-transmission problems under investigation include a study of the loading of cables in iron pipes, the impedance of the cable and heating of the pipe with various sizes of pipes and cables being measured. Cables under test are run in a special trench below the Laboratory floor to minimize variations in the ambient temperature. The heating of reinforcing steel in concrete near an alternating-current cable and the effect on the impedance of the cable is also being investigated, as well as the properties of cables close to steel bulkheads, as in marine work. The creep of cables in ducts is being studied. In connexion with the heating of buried cables, the thermal resistivity of soils is being investigated by determining the temperature-rise of a buried heater; the results obtained vary over a considerable range with different soils, and some of them are subject to seasonal variation.

The effects of corona discharge on dielectrics such as varnished cloth are being studied. In connexion with filling compounds for cable-joints, measurements are being made of the tensile and impact strengths of bitumen at low temperatures and of its elasticity and plasticity at normal temperatures. Long-period extension and recovery tests and tests at audible frequencies are of especial interest.

Harmonics in alternating-current transmission and distribution systems are being investigated from the point of view of possible telephone interference, especially where mercury-arc rectifiers are employed, the noise-voltage being studied by harmonic analysis or by measuring apparatus incorporating a weighting network.

An extensive study is being made of the corrosion of various metals which might be used for earth electrodes when buried in different types of natural soil and in salted soils. About a dozen sites are employed; specimens are dug up for examination at intervals, the longest period of test (uninterrupted) being 5 years. Certain metals appear to be satisfactorily resistant in most soils, but the behaviour of others varies widely. The soils from the various sites are chemically analyzed with a view to determining the principal factors governing rates of corrosion; a special method of estimating the exchangeable bases present in the soil is regarded as promising. Soil-analysis is also employed in connexion with the corrosion of lead cable-sheaths; photo-micrographs are employed

in connexion with the last-mentioned studies and for various other work such as the study of asbestos fibres and transformer-steels.

Much attention has been devoted to methods of measuring and suppressing radio interference; three measuring sets have been developed, covering respectively the long and medium wave bands, the 10–50 megacycle band, and the 20–100 megacycle band. The peak-voltmeter principle is employed for the measurement of ignition interference on ultra-short waves; the time-constants of the circuits employed are found to be of special importance, and suitable values are being determined. The properties of frequency-changer valves have been studied, since they are particularly important in noise-measuring equipment, especially as inter-modulation has to be minimized as far as possible. Considerable advances have been made in the design of high-frequency chokes for interference-suppression.

In connexion with high-voltage equipment, the intrinsic electric strength of dielectrics is being investigated over ranges of temperature between − 180° C. and 800° C. for crystals, minerals, and organic materials. Special methods of testing have been developed; in certain cases inconsistent results have been found to be due to breakdown of the ambient medium, and it has been possible to avoid that difficulty by selecting a medium of appropriate electrical properties. The results obtained from mica and crystalline salts show some concordance with the Fröhlich theory based on crystal-structure, but varnishes behave differently. In the case of varnish films their resistance to moisture-penetration is also being studied.

The properties of paper condensers are being examined, the breakdown voltage under various conditions of loading, the power-factor, the leakage and the effects of temperature-changes being studied.

The effects of electric currents passing between concrete and reinforcing bars are being investigated by embedding a steel rod in concrete, immersing the concrete in water and applying direct or alternating voltage between the rod and the water. With the rod positive, electrolysis causes rapid corrosion of the rod and destroys the bond, the expansion breaking up the concrete. With the rod negative, and with alternating current, the concrete surrounding the rod deteriorates and bond-strength is reduced. (This investigation has been undertaken in connexion with the use of reinforced-concrete poles for transmission-lines.)

A research on the effects of earthing electrical circuits to water-mains has been initiated in collaboration with a Committee of The Institution on which electrical and water-supply interests are represented. A comprehensive bibliography of previous work has been prepared, and an experimental programme is now being drawn up.

Various methods are employed for studying surge-phenomena on transmission-lines, of which the most important is the production of artificial surges by a 1,000-kilovolt surge-generator, the voltage and wave-

form of the surge at various points of the line being studied by a transportable high-speed continuously-pumped cathode-ray oscillograph. The effect of corona-loss in reducing surges with initial peaks above the corona-voltage has been studied. The distribution of surge-stress in the winding of a transformer terminating a line has been examined, and various methods of minimizing the concentration of stress on the first few turns have been compared. In addition to the initial peak stress, a surge of sufficient duration may initiate high-voltage resonant oscillations in the winding at supersonic frequencies. Lightning currents in overhead-line towers have been measured by means of magnetic links, and the occurrence and approximate nature of surge-voltages on transmission-lines have been recorded by means of klydonographs.

For experimental work on switchgear, a 10,000-kilowatt 6·6-kilovolt alternator is utilized; it is run up to speed by a 300-h.p. motor and can provide a short-circuit output of 150,000 kilovolt-amperes. Research carried on in this Laboratory has already led to the commercial development of the baffle type of oil circuit-breaker, and the development of the gas-blast breaker dates from 1926. The phenomena during the rupture of an arc are studied by a cathode-ray oscillograph, the time-sweep of which is tripped and reset by an "electron relay" developed from a cathode-ray tube, the control being so arranged that records are taken at about current zero of each successive half-cycle until the arc is broken. The contact-wear of air-break contactors at various rates of operation is also being studied in the switchgear laboratory.

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